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Load Testing and Rehabilitation of the Sioux Narrows Bridge

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Load Testing and Rehabilitation of the Sioux Narrows Bridge

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Abstract:	The Ontario Ministry of Transportation (MTO) has been actively engaged in the research and development of wood bridges since 1973. The Sioux Narrows bridge has played an important role in this work. Built of B.C. Douglas Fir in 1936 it supports a main clear span of 64 m and is believed to be the largest single span wood highway bridge in North America. Originally it was proposed that the bridge be replaced in 1975 but demolition was delayed following an extensive investigation by students who concluded that the structure could be rehabilitated. Subsequently MTO initiated an in-depth investigation, including inspection, analysis and field testing which confirmed that the existing bridge could be maintained. The objective of this report is to document the extensive work performed by MTO. This includes a detailed field inspection, numerous structural analysis and field testing, as well as a record of the successful replacement of the old nailed laminated deck with a new prestressed wood deck in 1982.
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Load Testing and Rehabilitation of the Sioux Narrows Bridge

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Acknowledgements

The move to preserve the Sioux Narrows bridge began in 1974 with a group of undergraduate students who were studying the bridge as the thesis requirement for their university degree. As with the many people who have since been involved with this majestic structure, the students were moved to become totally involved in the struggle to preserve this historic bridge.

I feel I would be remiss not to specifically acknowledge the contribution of P.W. Olynyk who originally identified the bridge as a potential University thesis project in 1973, after having visited it as an engineering student under the employ of MTC that same year. In addition, the feasibility of conducting the original student investigation and having the results administered by the MTC would not have been possible without the support and assistance of Mr. P.S. Csagoly the then Principal Research Officer, and Mr. K.L. Kleinsteiber, Head of Approvals at MTC.

During the subsequent years the work to preserve and rehabilitate the bridge has had periods of uncertainty. Considerable optimism was needed when considering the possibility of upgrading an old wooden structure to meet present day requirements. Throughout the work, a number of individuals have contributed extra time and effort to keep the structure intact until now: ten years after the original date of its proposed replacement. These are:

From the Ontario Ministry of Transportation and Communications:
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- B. Bakht, Principal Research Engineer, Research and Development Branch,
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- M. Poskovich, General Superintendent, Thunder Bay, and
- I. Trugnach, General Foreman, Thunder Bay.

1/ INTRODUCTION

The Ontario Ministry of Transportation and Communications (now called the Ontario Ministry of Transportation, or MTO) has been actively engaged in the research and development of wood bridges since 1973 [1]. The Sioux Narrows bridge shown in Figure 1 has played an important role in this work as it stands as a monument to past engineering achievements in wood bridge construction. Built of British Columbia Douglas Fir in 1936 it supports a main clear span of 64 m and is believed to be the longest single span wood highway bridge in North America.

The bridge was originally proposed for replacement in 1975 but it was delayed following an extensive student investigation [2] which concluded that the structure could be rehabilitated. Subsequently MTC initiated an in-depth investigation, including inspection, analysis and field testing which confirmed that the existing bridge could be maintained.

The objective of this report is to document the extensive work performed by MTC which led to the commencement of its rehabilitation in 1982. This includes a detailed field inspection, numerous structural analyses and field testing. Included also is a record of the successful replacement of the old nailed laminated deck with a new prestressed wood deck and the addition of a new wood sidewalk cantilevered outside the existing structure.

2/ HISTORICAL BACKGROUND

Until recently the life expectancy of wood bridges in Ontario was assumed to be 40 years, after which time the bridge would be arbitrarily replaced. Therefore in 1971, as the Sioux Narrows bridge approached the end of its so-called "life expectancy", it was put on a program for replacement scheduled for 1975.

However, this was occurring at a time when MTC's interest in wood bridges was being renewed. The Ministry's on-going bridge testing program began to include wood bridges in 1973. The results of these load tests posed serious questions as to the validity of the old "40 year rule" for wood bridges. Therefore in 1974 when a group of undergraduate students prepared a thesis in which they proposed an evaluation of the Sioux Narrows bridge, the MTC provided support. The student's proposal was to include an extensive site inspection and structural analysis.

The student investigation [2] included a detailed dimensional survey as no existing drawings for the bridge could be located. In addition, a number of decay investigation techniques were employed as directed by the Faculty of Forestry at the University of Toronto. Subsequently the trusses were analysed according to the Proposed Ontario Bridge Design Load [3] upon which the present Ontario Highway Bridge Design Code (OHBDC) [4] was based. The results of the student investigation indicated that the original bridge showed no signs of decay and according to their analysis it should be capable of carrying normal highway loads.

Subsequently the bridge's replacement was delayed and MTC initiated an extensive research project for evaluating the bridge with the prospect of rehabilitation. The work performed by MTC including the deck replacement and new sidewalk in 1982 is the subject of this report.

3/ SITE DESCRIPTION

The Sioux Narrows bridge shown in Figure 1 was built in 1936 with dense select structural Douglas Fir, pressure treated with oil-borne creosote. Located on Hwy 71 just south of Kenora, Ontario, it forms a critical link between the Trans-Canada Highways.

The structure has an overall length of about 110 m comprising a 9.1 m long south approach span, a 64 m main span and eight 4.6 m long north approach spans. Except for the laminated wood deck all the wood members are comprised of the original B.C. Douglas Fir.

3.1/ Approach Spans

All the approach spans were simply constructed of longitudinal sawn wood stringers supporting transversely laminated nailed wood decks.

The 9.1 m long south approach span has 140 mm x 440 mm stringers spaced at 460 mm centres as shown in Figure 2. These stringers support a 190 mm deep laminated wood deck. The 4.6 m long north approach spans utilizes 140 mm x 340 mm wood stringers and a 140 deep laminated wood deck as shown in Figure 3.

The south approach span is supported on a timber pile bent at its south end and directly on the main south concrete pier at its north end. The first north end span adjacent to the main span shares the support of the main north concrete pier. The remaining north end spans are supported on heavy timber trestles as shown in Figure 4. These trestles have 280 mm x 280 mm posts, caps and sills with 60 mm x 235 mm bracing.

3.2/ Main Span

The main span is a 64 m long "Box Howe" truss with wood main chords and diagonals, and vertical steel bar tension members. These trusses support transverse wood king post floor beams which bear directly on the bottom chords. The floor beams in turn support longitudinal sawn wood stringers and a transverse laminated wood deck similar to the approach spans.

3.2.1/ Trusses

Details of the member sizes are provided in Figure 5 with some overall dimensions given in Figure 6. Each main compression chord and the end diagonal are comprised of four 190 mm and 483 mm wood members bolted together. Four 190 mm x 533 mm wood members make up the bottom chord. The top chord is continuous through two 4.57 m panels as shown in Figure 6, the bottom chord is made continuous throughout its length by staggering butt joints in alternating pieces.

The main interior compression diagonals are comprised of three wood members; two outside pieces 289 mm wide and one in between at 184 mm wide. The 184 mm wide interior member is discontinuous at mid length where the opposing secondary diagonal passes through. The depths of the main diagonals vary from 394 mm at the ends of the span to 241 mm at the mid span. All secondary diagonals are one solid sawn member of 184 mm wide x 235 mm depth. All diagonals butt against steel thrust blocks which are keyed into the bottom and top chord members as shown in Figure 7.

Each vertical tension member is made up of three steel bars threaded at their ends. These bars project beyond the top and bottom chords and are anchored to heavy steel anchorage plates (see Figure 7) and vary in size from 70 mm diameter at the ends of the span to 35 mm diameter near midspan.

3.2.2/ Floor Beams

The king post beams are comprised of a wood beam member and centre post trussed by a pair of steel bars as shown in Figure 8. The wood beam is made up of five wood members all 337 mm deep. Two are 64 mm wide and three are 95 mm wide.

The king post is one solid sawn member and varies from 292 mm x 356 mm at the top to 178 mm x 279 mm at the bottom where it is capped by a steel shoe. This steel shoe supports a pair of 44 mm diameter steel bars which are trussed between the ends of the wood beam. The bars are anchored to heavy steel plates at the beam ends as shown in Figure 8.

The floor beams rest directly on the bottom chords of the trusses and are spaced at intervals of 2.29 m (7.5 ft.) along the bridge.

3.2.3/ Deck System

The deck system, supported by the floor beams, is comprised of 115 mm x 285 mm sawn wood stringers with a 140 mm deep transverse laminated wood deck as shown in Figure 8. The stringers are continuous over two spans and lapped at alternating floor beams as shown in Figure 7.

3.3/ Additional Information

The original wearing surface was made of 25 mm thick asphalt planks measuring 300 mm x 1200 mm nailed to the laminated deck. This form of surface was maintained until 1982 when the deck was rehabilitated. Information regarding the new prestressed wood deck and asphalt wearing surface will be detailed in Section 8.

All spans were provided with standard 150 mm x 150 mm wood curbs and 38 mm x 127 mm wood railings. These were to be replaced during the deck rehabilitation in 1982.

Additional information and details of the structure, including the new deck and sidewalk may be obtained from the MTC file under Site 41S-71 and Contract 81-226.

4/ SITE INSPECTION

In addition to the field inspection carried out by the student group in 1975 [2] a more detailed dimensional survey was carried out by MTC in 1977. Subsequently an additional inspection was performed by a specialized consultant in 1979 [5]. The latter inspection was directed primarily at decay, in order to ensure that the structure was worthy of rehabilitation. The inspection would also identify the wood species and material grade so that realistic material properties could be assigned in evaluating the structure's load carrying capacity.

Details of the inspection can be obtained from Reference 5, however, the following is a summary of the findings.

4.1/ Reference 5 Conclusions

From our visual examination, sampling and grading of the timber components, we concluded that with the exception of the deck, the structure is generally comprised of high quality (dense select structural grade) Douglas Fir and is in excellent condition. The few problems noted are identified by location in Figure 1 (of Reference 5) and are listed below.

1/ The untreated spruce-pine-fir deck is in the process of localized but active decay and some unsupported ends are being broken off between the stringers.

2/ The "X" bridging between the south span stringers is loose.

3/ The first three trestles from the north end have been damaged by the rock fill.

4/ The secondary diagonals in some locations are not fitted tightly to their end bearings.

5/ One of the railing posts is broken.

6/ Six of the top chord bracing struts contain pockets of decay and the tie rod bracing at this level requires adjustment.

7/ One of the stringers in the main span is of low grade material.

4.2/ Additional Observations

In addition to the findings of the consultant, two additional items regarding the bridge's rehabilitation required consideration.

The overhead diagonal portal frames at the entrances to the truss span can be seen in Figure 1. Such framing is provided at every second panel point along the trusses to resist side-sway. However, the end portal frames at the entrance to the trusses reduce the overhead clearance and also show signs of being hit by over-height vehicles. A rearrangement of this bracing was to be considered to increase the overhead clearance and reduce the chances of vehicular collisions.

Though the bridge is a two-lane structure, the roadway width is only 7.32 m (24 ft.) and therefore is quite narrow. This has not proved to be a problem with respect to vehicular traffic, but creates a hazard for pedestrians should two vehicles approach from opposite directions leaving no room for the pedestrian. Therefore a safe pedestrian access was to be considered when the bridge was being rehabilitated.

4.3/ Discussion

Each problem noted by the consultant will be discussed separately with respect to the action which was to be taken, as well as any additional details which related to that particular item.

1/ The untreated spruce-pine-fir deck replaced the original treated deck in 1971. This occurred at the time when rehabilitation of the bridge was not being considered. The untreated deck was considered a temporary measure in light of the bridge's proposed replacement scheduled for 1975.

Therefore prior to this most recent inspection, replacement of this untreated deck was already considered a requirement of the rehabilitation process.

2/ Cross bracing between wood stringers in a bridge is not currently considered an adequate form of diaphragm. Should a structural evaluation show that diaphragms are required, solid blocking would be provided according to the OHBDC [4].

3/ Further detailed inspection of the trestle problem at the north end was conducted by MTC. It was concluded that the displacement of the wood trestles at their bearing with respect to the concrete supports occurred at the time of construction during the placement of the rock fill. It was not considered to be a problem as the trestles still retain adequate bearing, and are extensively braced in both directions. However, it was recommended that the situation be kept under observation during regular inspection to ensure that it is not progressive.

4/ The effects of the lack of participation by these secondary members was to be evaluated during the structural evaluation. Should the evaluation show that these members are critical to the structural capacity they would be shimmed to reinstate their participation.

5/ The overall condition of the railings and posts was considered poor. New materials were to be installed during rehabilitation.

6/ The decayed bracing would be evaluated to determine whether they should be replaced or whether the strength remaining in them was still adequate. The extent of decay was minimal and could be arrested by proper chemical treatment.

The overhead steel cross bracing would be tightened underneath the structure when rehabilitation work on the trusses was to be performed.

7/ This lower grade stringer would be reinforced or replaced if the evaluation indicated that it did not provide adequate load carrying capacity.

4.4/ Recommendations (1980)

The 1979 field inspection [5] indicated that the structure did not have any advanced decay and therefore could be considered worthy of rehabilitation. According to the original structural evaluation performed by the student group in 1975 [2] the main truss span appeared to be of adequate strength to carry the everyday loads. A brief review of the approach spans showed the structural arrangements met the current MTC standards for sawn wood stringer bridges. Therefore it was decided that the bridge would be rehabilitated.

Because of the above, and the fact that the untreated laminated deck was showing signs of accelerated deterioration, it was decided that the deck should be replaced as soon as possible. At the same time, the pedestrian hazard and the overhead portal frame problem were to be considered as well.

Tentatively, the deck replacement, the addition of a new wood sidewalk to be cantilevered outside the exiting structure, and the modification of the overhead portal framing was scheduled for the fall of 1982.

Concurrently a major MTC research project was initiated which would include a detailed structural evaluation based on extensive structural analysis, field testing and possibly model testing of the bridge. In addition to aiding in the rehabilitation of the Sioux Narrows bridge, the information generated by this work would be valuable in evaluating many other similar wood truss bridges which exist in Ontario.

In summary, only the deck, sidewalk and portal frames were to be considered in the scheduled rehabilitation. Any upgrading of other structural items was to be left until the detailed evaluation was completed, unless in the interim, and prior to the reconstruction work in 1982, the evaluation indicated a deficiency which could be dealt with during the scheduled work.

5/ PRELIMINARY STRUCTURAL EVALUATION

A preliminary structural evaluation was performed on the bridge in 1980. It was limited in extent as the first edition of the OHBDC [6] was not fully functional in the area of wood bridges. However, for the purposes of presentation this evaluation has been expanded upon using the new edition of the OHBDC [4]. This evaluation assumes ideal conditions with respect to the member performances and load distribution (as would have been assumed for design purposes). Member resistances are based upon the identification of the wood materials as dense select structural Douglas Fir for all members except the deck. The steel bars are assumed to be structural grade.

The report does not include all the calculations, but does include any critical assumptions and in some cases the resulting load distribution criteria. Primarily, comparisons are made between the factored load effects and the factored member resistances. The applied member resistance formula are presented in sequence of application as a means of discussing critical assumptions relating to such items as load sharing, load duration and the like.

5.1/ Member Resistances

Prior to presenting the results of the evaluation it is necessary to discuss the selection of member strengths relating to the original Douglas Fir members in the bridge.

The structure was built in 1936 and therefore the heavy Douglas Fir timbers are assumed to have been obtained from virgin forest stands commonly referred to as "first growth" material. However, for the last several decades Douglas Fir timbers have been procured from second and third growth materials from forests which have developed after the original logging operations. This material appears to be of lower quality and therefore lower strength when compared to the first growth material available in the early 1900s [7]. In light of this it is necessary to consider some adjustment to the current design strengths to compensate for the superior quality of the timbers used in the Sioux Narrows bridge.

In subsequent discussion this adjustment will be referred to as the "adjustment for age".

In addition to age, the Douglas Fir timbers in the Sioux Narrows bridge also exhibit another quality not currently recognized by the OHBDC specifications. Under the NLGA [8] grading rules, material meeting certain grain concentrations is graded as "dense". In the past, such material has been given higher allowable strengths based upon the fact that it had more material per unit volume and therefore higher strengths.

However, this higher strength was derived from the testing of small, clear, straight-grained specimens in which the effects of natural defects such as knots and slope of grain were not directly apparent. The results from such tests would be adjusted, to account for the effects of the natural defects allowed under each specific grade when deriving the allowable design strengths. However, recent in-grade (full size) testing [9], [10] has indicated that at the lower end of the strength distribution, at the level from which design strengths are derived, failure may be governed primarily by natural defects. Therefore, the effects of such variables as density, moisture content and size may not be so apparent. This phenomenon would especially govern the strengths of lower grade materials in which more natural defects are allowed. It is necessary to consider this when deciding on an adjustment of an assumed strength for the dense material contained in the Sioux Narrows bridge. The effects of density on the strength of these timbers is probably not as significant as has been assumed in the past. However, since the material here is of very high quality, with few defects, some increase in strength for density will be considered.

In summary, there are two adjustments that will be made to the current specified strengths contained in the OHBDC for select structural Douglas Fir. One is for age based on its procurement from first growth material and the second is for density. Consideration for density will be limited, based upon its assumed reduced effect at the lower end of the strength distribution.

In order to derive some relative factors for adjustment, Table 1 contains the specified allowable stresses for select structural Douglas Fir as published in a number of AASHTO specifications [11] from 1953 to 1983. Comparing the allowable stresses in the current AASHTO specifications with those from the earliest edition one can see a marked reduction in the published values over that 30 year period. In addition, all of the specifications offer a definite premium to material graded as "dense".

Table 2 contains the specified strengths for select structural Douglas Fir as published in the OHBDC [4] and the CSA-086 [12] both of which utilize Limit State Design (LSD) methodologies. Since the LSD approach is based upon Ultimate Strength Design (USD), a direct comparison with the previous allowable stresses, which are used in Working Stress Design (WSD), cannot be made. Therefore using the comparative values in Table 1 the specified strengths in Table 2 will be adjusted for age and density. These strengths will be adjusted first for age using the relative change noted between the values published in the 1953 and 1983 editions of AASHTO. Subsequently the result will be adjusted for density according to the differences noted in the 1953 edition of AASHTO, however, only half of the relative differential will be applied, in order to account for the assumed reduced effect of density at the lower end of the strength distribution.

5.1.1/ Specified Strength in Bending

According to Table 1 the allowable bending stress for select structural was about 18.8% higher in 1956 than in 1983. In addition, dense material was allowed a 13.2% premium over normal weight in 1956. According to previous discussion the specified bending strengths from the OHBDC and CSA-086 would be increased by about 18.8% for age and about 6.8% for density. This results in a combined increase of about 26.5% and provides assumed specified bending strengths as listed in Table 3.

5.1.2/ Specified Strength in Compression Parallel to Grain

Applying the same approach for the strength in compression parallel to the grain, the adjustment for age would be about 31.8% and about 3.4% for density. This results in a combined increase of about 36.4% providing

assumed specified compressive strengths parallel to grain as listed in Table 3.

5.1.3/ Specified Strength in Tension Parallel to Grain

The specified strength in tension parallel to the grain requires a little more discussion. As mentioned previously, the allowable stresses in the past were derived from tests on small, clear specimens. Such testing would invariably show that tension parallel to the grain is the highest strength mode in wood. However, tension strength is also the most sensitive to the effects of the natural defects in full size members. Therefore, in order to accommodate for the more prominent effects of these natural defects on the tensile strength, the latter was assumed to be equal to the bending strength. This approach was believed to be conservative as the small clear specimen testing showed the tensile strength much higher than the bending, while at the same time, the bending strength in full size members would also be governed by natural defects on the tensile face.

More recently it became evident that the tensile strength was even lower than the bending strength as can be seen in Table 1. This is believed to be due to the fact that in axial tension the entire cross-section is loaded in tension while in bending, only the extreme fibres are highly stressed in tension. Therefore, the latter would be affected by only those defects near the tension face, while the preceding condition would be sensitive to any and all defects.

In any case, the allowable tensile stress in Table 1 has only limited value for our purposes. The values from the earliest editions of AASHTO must be ignored. However, though the magnitude of the strengths between the bending and tension modes may be different, the relative change in magnitude over the years may be similar. This is based upon the fact that the failure mode of both is dependent upon the tensile strength. Therefore, it is assumed that the same adjustment applied to the bending strength, as determined in Section 5.1.1 may be applied to the specified strengths in tension. The assumed specified strengths for tension parallel to the grain are as listed in Table 3.

5.1.4/ Specified Strength in Horizontal Shear

Applying the same approach to horizontal shear as was used for bending, the adjustment for age would be about 41.1% and about 10.4% for density. However, in recent years it appears that a premium for density has not been provided. This would be understandable as the horizontal shear strength is primarily dependent on the weak material between the denser annual rings (visible grain). Therefore a denser wood does not necessarily produce a higher strength. Considering this, only the 41.1% adjustment for age was applied to the specified strengths in horizontal shear as listed in Table 3.

5.1.5/ Modulus of Elasticity

The modulus of elasticity does not appear to have varied drastically as can be seen in Table 1. Therefore no adjustment was considered for the values listed in the current standards.

5.2/ Evaluation of Deck and Stringers

The evaluation of the laminated deck and stringers in the approach spans and main span was conducted according to the current OHBDC [4] specifications. A sample design procedure including calculations can be found in Reference 13.

5.2.1/ Transversely Laminated Wood Decks

All of the stringer spacings on the Sioux Narrows bridge are 457 mm on centres. Therefore the assumed design deck span would be 400 mm derived from the "clear distance between stringers plus 1/2 stringer width" as specified in the OHBDC. This occurs in the main span where the shallowest deck depth also occurs. Figure 9 is a graph which was developed as part of an extensive calibration study [14] using the OHBDC. It shows that for the 140 mm deep deck the allowable span would be in excess of 500 mm for any species considered. Therefore, for all practical purposes further evaluation of the laminated deck on the Sioux Narrows bridge is not required. This assumes at this time that the exisiting untreated

deck would be replaced with a new treated deck of equivalent depth and orientation.

5.2.2/ Sawn Wood Stringers

The load distribution between stringers was determined according to the Simplified Methods as prescribed in Section 4 of the OHBDC. The member resistances were determined according to Section 13 of the OHBDC and subsequently adjusted for [illegible] factors including the previously discussed adjustments to the material strengths.

5.2.2.1/ Factored Load Effects

The total factored load effects for moment and shear for the stringers in all spans are listed in Table 4. The distribution widths (D_d) for live load moment and shear as listed in Table 5, were determined according to the simplified methods under sections 4.6 and 3.7 of the OHBDC. Only the ULS combination one was considered including dead load and live load. The assumed Dynamic Load Allowance (DLA) was 0.3 reduced by a factor of 0.7 as prescribed by Clauses 2.4.3.2.7 and 2.4.3.2.10 of the OHBDC.

5.2.2.2/ Factored Moment Resistances

The factored moment resistances as listed in Table 4 were determined according to the following:

$$M_u = \phi f_{bu} S k_d k_l k_m k_s \qquad \text{(Clause 13.6.1, OHBDC)}$$

where: ϕ = performance = 0.9

f_{bu} = specified strength in bending (MPa) (Tables 2 and 3)

S = section modulus (mm^3)

k_d = duration of load effect = 1.0, dead load alone not critical

k_l = modification for lateral stability = 1.0, support provided by deck

k_m = load sharing effect = 1.1, stringer spacing < 600 mm

k_s = size effect according to Clause 13.6.2.1 of the OHBDC.

According to Table 4 all moment capacities of the existing stringers would not meet the OHBDC requirements. However, if the specified bending strength is adjusted for age and density, all but one of member resistances become satisfactory.

In south approach span the moment resistance was still only 83% of the factored load effect. This deficiency was to be considered during future work.

5.2.2.3/ Factored Shear Resistances

The factored shear resistances as listed in Table 4 were determined according to the following:

$$V_u = \frac{\phi f_{vu} b d}{1.5} k_d k_m k_v$$

where: ϕ = performance factor = 0.8

f_{vu} = specified shear strength (MPa) (Tables 2 and 3)

b,d = width and depth of stringer (mm)

k_d = duration of load effect = 1.0, dead load alone not critical

k_m = load sharing effect = 1.1, stringer spacing < 600 mm

k_v = size effect = $\frac{45}{\sqrt{d}}$ for new select structural

or = $\frac{75}{\sqrt{d}}$ for existing select structural where members have no end splits (see discussion below)

The detailed inspection [5] revealed that the Douglas Fir members in the Sioux Narrows bridge were in excellent condition. It should be noted that in Clause 13.7.1.2.2 of the OHBDC measured end splits may be used to determine k_v using the following:

$$k_v = \frac{75}{\sqrt{d}} \quad \frac{1}{1 + 2(a/d)}$$ (Clause 13.7.2.2.2 OHBDC)

where:

a = length of end split, measured from the centerline of support to tip of the split, therefore, assuming end splits did not advance past the centerline of bearing, the value "a" would be zero.

According to Table 4 all the shear resistances may be considered borderline with respect to the OHBDC. However, if the specified shear strength is adjusted for age and density, all the shear resistances become satisfactory. Alternately, if the size effect factor (k_v) is adjusted to reflect no end splits, the shear resistances are even higher. Therefore the shear resistance of all the sawn wood stringers was considered to be adequate.

5.3/ Evaluation of King Post Floor Beams

The OHBDC does not provide for the distribution of axle loads to transverse floor beams by simplified methods, except where a continuous concrete deck is present. Therefore some conservative assumptions were made as will be described in Section 5.3.1.

The floor beams were evaluated for bending and axial compression in the wood beam, tension in the steel bars, and compression in the wood post.

5.3.1/ Factored Load Effect

The floor beam as shown in Figure 10 was represented by a two dimensional frame analysis [15] in order to determine the individual member forces. The distribution of axles along the bridge between floor beams was determined under two conditions as shown in Figure 11. Initially the stringers were assumed to provide the "as designed" continuity over two spans. Subsequently the individual spans between floor beams were assumed to be simply supported, representing the actual condition in which very little "hold down" from the stringer reaction is provided. In both situations the distribution of wheels along the bridge were taken

along straight lines representing no transverse distribution. These distributed loads were then introduced into the floor beam frame analysis as equivalent wheel loads directly on the floor beam as shown in Figure 12. Two vehicle positions were considered as shown in Figure 12 limited by the lane and vehicle widths prescribed by the OHBDC. Position 1 represents the vehicles as close to the centreline of the roadway as possible thereby generating the highest forces in the king post and steel tension bars. Position 2 represents the vehicles as close to the curbs as possible, under which a preliminary evaluation indicated the maximum bending occurs in the wood beam. The resulting maximum forces and subsequent factored load effects are given in Table 6. The assumed dynamic load allowance was the same as for the wood stringers as described in Section 5.2.2 of this report.

5.3.2/ Wood Beams

The factored axial compressive and bending resistances of the wood beam in the king post floor beams are given in Table 6. The bending resistance was determined in the same way as for the stringers in Section 5.2.2 of this report except the various modification factors were chosen to reflect the new conditions.

The factored axial compressive resistance was determined according to the following:

$$P_u = \phi\, A f_{pu}\, k_d\, k_m \quad \text{(Clause 13.8.1 OHBDC)}$$

where:

ϕ = performance factor = 0.9

A = cross-section area (mm^2)

f_{pu} = specified compressive strength parallel to grain (Tables 2 and 3)

k_d = duration of load effect = 1.0 (dead load alone not critical).

k_m = load sharing = 1.1, five pieces laminated together (full lateral support to the beam was assumed).

The results in Table 6 indicate that for the continuous stringer assumption and the resistances according to the OHBDC, the individual forces are less than the resistances. However, considering the interaction:

$$\frac{P}{P_u} + \frac{M}{M_u} \leq 1.0$$

yeilds an unsatisfactory value of 1.27. Even considering the simply supported stringer assumption for axle load distribution, the interaction would be 1.13. However, using the adjusted specified strengths to determine the member resistances the two preceding interactions would be 0.98 and 0.87. Therefore the combined bending and compression resistances of the wood beam in the king post floor beam was considered to be adequate.

5.3.3/ Wood Post

The factored axial compressive resistances for the post were derived as before except the load sharing factor k_m was equal to 1.0. Slenderness was not considered critical.

A direct comparison of the factored load effects and resistances in Table 6 indicates that the factored axial compressive resistance of the post was adequate.

5.3.4/ Steel Bars

The factored axial tension resistance of the steel bars was determined as follows:

$T_r = \phi A_n F_y$ (Clause 10.7.2 OHBDC, the alternate formulae, were also checked.)

ϕ = performance factor = 0.9

A_n = net section area at thread (mm^2)

F_y = assumed yield strength (MPa)

The values listed in Table 6 do not reflect any consideration for deterioration of the steel. Accordingly the evaluation indicated that the steel bars are considerably under strength and were to be considered during any future work.

5.4/ Evaluation of Main Trusses

The main span trusses were analysed for the factored OHBDC loads using a frame analysis [15]. The member resistances were determined by the methods prescribed in the OHBDC along the same lines as previously presented. Member resistances for the wood members were also determined according to the CSA-086 [12]. The latter was performed as a comparative approach for several reasons. The method for uniaxial bending in the OHBDC uses moment amplifications to account for the effects of slenderness in compression members. This procedure was adopted from the approach used in the design of concrete columns. The method for uniaxial bending in the CSA-086 follows the same procedure for slenderness as has been used in the past for the design of wood compression members. This represented an opportunity to compare the new OHBDC method with the previously accepted method while still working in the limit states design format.

In addition, the OHBDC was expected to produce conservative member resistances for wood tension members. The specified strengths in tension in the OHBDC are based upon limited test data and this is reflected in the specified strength values and the performance factor. This conservatism was accepted by the OHBDC considering the limited application of wood tension members in bridge construction. The parallel use of CSA-086 specifications is considered to provide more realistic tension resistances for the evaluation.

5.4.1/ Analysis Assumptions

The truss was analysed as a frame as shown in Figure 13 and the member end conditions were selected based on the actual circumstances. The top chord was assumed to be pinned at every second panel point where it was discontinuous. The main compression diagonals and secondary diagonals were assumed to be pinned at both ends to represent lack of moment transfer as shown in Figure 7. The bottom tension chord was assumed to be continuous throughout.

Dead loading from the deck system and live loading was assumed to be applied at the floor beam locations on the bottom chord. Dead loads of the truss members and bracing were to be applied at respective joints.

The live load analysis was performed initially by generating influence lines due to unit loads applied at each floor beam location for all members. Subsequently, the influence lines were used to determine the critical OHBDC truck position for each member. Both the OHBDC truck loading and lane loading were considered. Only the maximum factored force effects will be presented in this report and each will be identified as to whether the truck or lane loading governed.

The main truss was analysed under two conditions based upon the results of the field inspection [5]. Initially it was analysed assuming ideal conditions with all members participating in the structural response as shown in Figure 13. Subsequently, the analysis was repeated with all the secondary diagonals removed as indicated by the dotted lines in Figure 13. The latter was done for two reasons. The field inspection indicated that a good number of the secondary members were not engaged against the steel thrust blocks at the truss joints. In addition, the initial analysis under ideal conditions indicated that these secondary members would nearly always be in tension. Therefore, unless they were preloaded in compression they would not participate in the structural response of the truss.

5.4.2/ Factored Load Effects

Table 7 summarizes the factored load effects from the frame analysis for some of the primary members. The members listed represent an adequate range for the purposes of presentation. The member numbers are identified in Figure 13. The results from these two analyses are presented as representing the structure with secondary member performance and without.

Comparing the total load effects from both analyses, there appears to be a range of differences for different members. Members 2, 32, and 34, which are the end diagonal, top chord and bottom chord, respectively, do not appear to be affected by the removal of the secondary members. This is because the overall moment, assuming the truss to be a simple beam, would be the same in both instances. Therefore, since the overall cross section of the truss does not change, the member forces in the top and bottom chords would not change. However, the internal members of the truss demonstrate considerable differences in member forces under the two analytical assumptions.

Members 5, 10, 15, 20, 25, 30 and 35 represent the vertical steel bars from the end panel to the centreline of the truss respectively. With the removal of the secondary members, the forces in these steel tension bars increase dramatically as they are called upon to carry a large component of the tension forces which the secondaries would normally be assumed to carry. The increase in the bar forces is highest closer to the bridge centre line, with the force in member 30 being increased by a factor 4.4 due to the removal of the secondary members.

The redistribution of internal forces also increases the forces in the primary diagonals, though to a much lesser degree. The maximum increase occurs in member 23 by a factor of 2.0.

Considering the apparent lack of the secondary member performance as observed during the field inspection, as well as the fact that these members cannot perform in tension, the analysis results based upon the removal of the secondary members were to be used in evaluating the truss capacity.

5.4.3/ Member Resistance

A number of resistance formulae from the OHBDC have already been presented in previous sections. The subsequent discussion will show only those formulae from the OHBDC that have not already been presented adequately but are required for the truss evaluation. In addition, the comparative formulae from the CSA-086 will be provided.

5.4.3.1/ Wood Compression Members (Uniaxial Bending) OHBDC

According to the OHBDC all wood compression members must be evaluated for minimum end eccentricities, bow, and crook, so that pure compression is never considered. The member resistance is based upon the following interaction:

$$\frac{P}{P_u} + \frac{M_c}{M_u} \leq 1.0$$

where: P = total factored axial load

M_c = amplified factored moment

M_u = factored moment resistance, determined as presented previously

P_u = factored resistance in compression, to be used only in the interaction equation.

and, $P_u = \phi A f_{pu} k_d k_m$

where: ϕ = performance factor = 0.9

A = cross-sectional area (mm^2)

f_{pu} = Specified strength in compression parallel to grain (Tables 2 and 3)

k_d = duration of load effect = 1.0

k_m = load sharing effect, dependent upon number of laminations.

Slenderness

The effects of slenderness are accounted for in the amplification of the applied moments:

$$M_c = \delta M_o$$

$$\delta = \text{amplification factor} = \frac{C_m}{1 - P/P_{cr}}$$

where: c_m = factor relating the actual moment diagram to an assumed uniform diagram. In this evaluation this factor was conservatively assumed to be 1.0.

P = factored axial load

$$P_{cr} = \text{factored Euler buckling capacity} = \frac{\phi \pi^2 E_{05} I k_d k_m}{(kL_u)^2}$$

and, M_o = total factored maximum moment including the effects of end eccentricity, bow and crook.

5.4.3.2/ Wood Compression Members, CSA-086

According to CSA-086, slenderness in compression members is considered directly in the determination of the axial member resistance. The slenderness of the member is compared against specific limitations by which the column is then designated as short, intermediate or long.

Slenderness

$$C_c = \frac{L_e}{d} = \text{highest value considering both directions.}$$

where for our purposes:

L_e = effective length of the member in the plane of the truss

d = depth of member in the plane of the truss

$$C_k = \sqrt{\frac{0.9\ E_c\ K_{SE}}{F_c k_D\ K_{SC}\ K_H}}$$

where: E_c = modulus of elasticity for compression member = 0.74E (MPa)

E = modulus of elasticity (Table 3) (MPa)

F_c = specified compressive strength (Tables 2 and 3) (MPa)

k_D = duration of load effect = 1.15, the accumulated live load effect on bridges is considered equal to 1 month duration ([6], Clause C13.5.3).

K_{SE} = service condition factor for E_c, = 1.0

K_{SC} = service condition factor for F_c, = 0.91, wet service

K_H = load sharing factor, dependent on number of laminations and force mode considered.

Short Column $C_c < 10$

$$Pr = \phi\ F_c\ A\ k_D\ K_{SC}\ K_T\ K_H\ K_C$$

where: ϕ = 0.75

A = cross-sectional area (mm^2)

K_T = treatment factor = 1.0

K_C = slenderness factor = 1.0

Intermediate Column $10 < C_c < C_k$

$$P_r = \phi\ F_c\ A\ k_D\ K_{SC}\ K_T\ K_H K_c$$

where: ϕ = 0.75

$$K_c = 1 - 1/3\ \left(\frac{C_c}{C_k}\right)^4$$

Long Column $C_k < C_c < 50$

$$P_r = \frac{\phi \pi^2 E_c A}{12 C_c^2} K_{SE} K_T$$

where: ϕ = 0.55

5.4.3.3/ Wood Tension Members OHBDC

The factored tension resistance was determined from the following:

$$T_u = \phi f_{tu} A k_d k_s k_m$$

where: ϕ = 0.6

f_{tu} = specified tension strength (MPa) (Tables 2 and 3)

All other factors as before.

5.4.3.4/ Wood Tension Members CSA-086

Factored tension resistance was determined from the following:

$$T_r = \phi F_t A_n k_D K_{st} K_T K_{zt} K_G K_H$$

where: ϕ = performance = 0.7

F_t = specified tension strength (MPa) (Tables 2 and 3)

All other factors as before except:

K_{st} = service condition factor for F_t, = 1.0

K_{zt} = size effect factor tension = 1.0

5.4.3.5/ Wood Bending Members CSA-086

Factored bending moment resistance was determined from the following:

$$M_r = \phi F_b S k_D K_{sb} K_T K_{zb} K_{Gb} K_H K_L$$

where: ϕ = 0.7

F_b = specified bending strength (MPa) (Tables 2 and 3)

S = section modulus (mm^3)

All other factors as before except:

K_{sb} = service condition factor for F_b, = 1.0

K_{zb} = size factor = 1.0

K_{Gb} = specific grade factor for bending = 1.0

K_L = lateral stability factor = 1.0, depth less than width.

5.4.3.6/ Horizontal Shear CSA-086

The factored horizontal shear resistance is required to evaluate the shear induced in the bottom chord due to the keyways of the steel thrust blocks which support the truss diagonals. The k_v factor in the formula for resistance in the OHBDC must be present as the specified shear strengths were derived based upon calibration to this factor. However, for the shear condition introduced by the thrust blocks this k_v factor cannot be properly interpreted. Therefore the horizontal shear resistance of the wood was determined using only the CSA-086.

The factored shear resistance was determined from the following:

$$V_r = \phi F_v A k_D K_{SV} K_T K_H K_N$$

all factors as before, except:

A = area of shear plane being considered (mm^2).

5.4.4/ Discussion on Factored Loads and Resistances

Tables 8 through 11 provide the comparison of factored load effects and factored member resistances for the truss members. The factored load effects are those determined from the analysis which ignored secondary member performance.

5.4.4.1/ End Diagonal

The end compression diagonal, identified in Figure 13 as member 2, was assumed to have an effective length equal to the distance between the intersections of its neutral axis with those of top and bottom chords. The results of the evaluation, using the gross cross-sectional area, are presented in Table 8.

Considering both the OHBDC and the CSA-086 without the adjustment of the specified strengths as described in Section 5.1, this member appears to have adequate capacity. Because of the higher specified strengths due to age and density of the material, the member displays considerable reserve strength under the OHBDC loadings.

5.4.4.2/ Top Chord

The main compression chord, identified in Figure 13 as member 32, was assumed to have an unsupported length equal to the distance between the truss joints through which it passes. The effective length was determined by multiplying this unsupported length by a factor of 0.8. This represents the member assumed to be pinned to one end, and fixed against rotation at the other. The results of the evaluation, using the gross cross sectional area are presented in Table 8.

Taking both the OHBDC and the CSA-086 into consideration, without the adjustment of the specified strengths, the top chord displays inadequate member capacity. However, using the adjusted specified strengths given in Table 3, the top chord satisfies the interaction equation. Assuming these specified strength adjustments are realistic the top chord would have adequate capacity.

5.4.4.3/ Interior Diagonals

The primary internal compression diagonals, identified in Figure 13 as members 13, 23 and 33, were each evaluated under two support assumptions. Initially the effective length was assumed to be the distance between the points of intersection of each members neutral axis with those of the top and bottom chords. This is a somewhat conservative assumption, as the secondary diagonals which pass through the primary members at mid-length provide some stabilizing support though they are not properly engaged at their ends. Alternatively, the evaluation was also performed assuming

the secondary members do provide buckling support to the primary diagonals at mid-length. The latter results in a reduction of the effective length to one-half the value initially assumed. This current discussion of secondary diagonal performance should not be confused with the previous discussion relating to the overall structural response of the truss. The secondary member are still not assumed to participate in the structural analysis, they are only assumed to provide support against in-plane buckling to the primary diagonals.

In general, the results in Tables 9 and 10 indicate that without support at mid-length the primary diagonals do not have adequate capacity. The large discrepancy between the OHBDC and CSA-086 interaction values in Table 10 for member 33 is due to the OHBDC's treatment of compression members. As previously mentioned, in slender members the moment amplification approach is only applicable to the members with reasonably square sections. In this case the width to depth of the member is over 3:1 and therefore only the values derived using the CSA-086 should be considered applicable.

If support at mid-length was provided by the secondary members, the primary diagonals members would have adequate capacity with considerable reserve strength.

5.4.4.4/ Bottom Chord

The bottom chord tension member, identified in Figure 13 as member 34, was evaluated at various net sections according to a number of bolt patterns and splice details. The results of the evaluation are given in Table 8. As predicted previously, the OHBDC specifications produced much more conservative member resistances for this wood tension member than the CSA-086.

If only the results from the CSA-086 in Table 8 are taken into consideration, the member does not have adequate resistance even using the adjusted specified strength values. The latter produced the lowest interaction value, 1.10. The consequences of failure of the bottom chord is such that this 10% overstress must be reviewed closely and therefore this problem was to be given serious attention during any future work on the trusses.

5.4.4.5/ Vertical Steel Tension Bars

The vertical steel bar tension members are identified in Figure 13 as members 5, 10, 15, 20, 25, 30 and 35. These members were evaluated using only the OHBDC specifications. The material was initially assumed to be structural steel with a yield strength of 230 MPa. Subsequently it was assumed to be a reinforcing steel of intermediate grade, with a yield strength of 275 MPa.

The results of the evaluation, using the net section at the threaded end of the bars, is given in Table 11. None of the members have adequate capacity when compared to the factored load effects where the secondaries member performance was ignored. When compared to the analysis where the secondary member performance was included, all the members would have had adequate capacity. However, the latter comparison was made only as a means of demonstrating that, given the ideal conditions which may have been assumed for the original design, the members would have been acceptable. The deficiency of these steel bars was to be seriously considered during any future work on the trusses.

5.4.4.6/ Horizontal Shear Bottom Chord

The critical horizontal shear force was determined to be at the end diagonal where the last steel thrust block is located. The diagonal member force is the largest, while the effective shear plane in the wood supporting the keyways is the smallest. The horizontal shear was taken as the horizontal component of the factored axial compressive force in the truss diagonal as indicated in Figure 14. The shear plane in the wood was taken over its full length from the innermost keyway as shown.

The results of the evaluation are given in Table 12. Shear in the steel keyways and bearing on the wood are not a problem. However, horizontal shear at the end of the bottom chord appears to be critical. This condition was to be given further consideration during any future work on the trusses.

5.4.5/ Summary Truss Evaluation

Table 13 summarizes the evaluation results of Tables 8 through 12 in terms of the interaction values. In the case of single force modes, such as tension in the vertical bars, the tabulated value is the factored load effect divided by the factored member resistance.

The last two columns in Table 13 represent the adjusted specified wood strengths and the higher steel yield strength. The overstress conditions are greatly reduced such that only members 34, 10, 20, 25, 30 and the horizontal shear in the bottom chord display inadequate member resistances.

The underlined values in Table 12 are believed to be the most realistic for current evaluation purposes.

The member resistances of the wood compression members may be best determined using only the CSA-086 specifications. None of these members are reasonably square sections and so the validity of applying the moment amplification approach from the OHBDC is questionable. The same applies for the bottom chord in tension and shear because the OHBDC is conservative with respect to the determining factored tension resistances and because it is impossible to interpret the k_v factor for shear. The member resistances for the steel bars should be based upon the lower yield strength as there is presently no data to justify a higher value, though material testing may be considered.

5.5/ Summary of Preliminary Evaluation

In general, the evaluation indicated that the overall structural capacity of the Sioux Narrows bridge is remarkably good considering its age and the fact that it was engineered with a design load suitable for the 1930s. However, in most instances an acceptable structural capacity was possible only when the age and density of the original wood was taken into consideration.

The only wood members which, using the adjusted specified strengths, still display inadequate resistances are the stringers in the south approach span and the bottom chord of the main trusses. The former has inadequate moment resistance and the latter have inadequate tension and shear strengths. The adequacy of the interior primary compression diagonals of the trusses is also questionable if the secondary members do not provide them with support against buckling. This support can easily be achieved, however, by shimming the secondary members at their ends.

All of the steel tension bars utilized in the bridge have inadequate member resistances. The overstress condition in the floor beam bars, as indicated in Table 6, is as much as 92%. In the main truss the overstress ranges up to 51%.

It is possible that considerations of loading and/or load distribution adjustments could bring most of the wood members within acceptable levels. Such adjustments may be considered at a later date. However, it is not possible to satisfy the deficiencies of the steel components without radical changes to the structural system or material strength. Therefore correction to these members should be the foremost consideration of any plans for upgrading of the existing structure.

In general, the results of previous site inspections and the preliminary structural evaluation indicated that the bridge should be slated for rehabilitation. The cost of the replacement of the steel bars and reinforcement of the bottom chord of the truss could not outweigh the high costs of a bridge replacement. To provide a better understanding of the structural response of the bridge and possibly eliminate concerns raised by some of the conservative results from the evaluation, load testing of the bridge was recommended. Load testing was scheduled for the spring of 1982 and any subsequent rehabilitation for the winter of 1982/83.

6/ PROPOSED TESTING AND REHABILITATION PROGRAM

The load testing of the structure was originally scheduled for late 1982 or early 1983. Testing was to include the main span deck system, trusses, and approach spans. The objective was to evaluate the responses of the various structural systems and refine the analytical assumptions used in the preliminary evaluation. Subsequently a re-evaluation would be performed to determined more precisely the level of deficiency of the bridge and to develop proposals for methods of reinforcement.

As well, construction and testing of a 1:12 scale model of the main truss span was to be performed. This was done both to support the on-going analysis by reducing the need for extensive field testing and to evaluate the effectiveness of model testing.

6.1/ Existing Laminated Deck Deterioration

The original program was disrupted by the alarming increase in the rate of deterioration of the existing laminated deck as shown in Figure 15. By the fall of 1981, large areas of the deck displayed localized failure due to advanced decay. Therefore the decision was made to schedule a deck replacement for the following spring, regardless of the status of the research and development work.

6.2/ Model Testing

The decision to move the deck rehabilitation program forward caused difficulties in the 1:12 scale model testings. The construction of the model was very slow and it was unlikely that it would produce results in time for their application in the deck rehabilitation.

However, the model testing was to continue in an effort to evaluate the effectiveness of scale testing wood bridges. If successful, some of the data could then be used during subsequent evaluations of the Sioux Narrows bridge. The model testing has been treated as a separate project and results will probably be given in another report.

6.3/ Preliminary Load Testing of Main Span Deck

According to the preliminary structural evaluation, the critical members in the deck system were the king post floor beams in the main span deck. It was considered to be most appropriate to upgrade the floor beams during the replacement of the laminated deck. Therefore the schedule of the research and testing program was further modified to include a preliminary testing of the existing main span deck in the fall of 1981 prior to the deck replacement. The test results would be quickly evaluated in an effort to produce an analytical model which could simulate the structural response of the deck system. Subsequently this model would be used to derive a deck replacement program ready for spring of 1982, with the possibility of correcting the deficiency in the king post floor beams at this time.

6.4/ Summary

It appeared that field testing of the bridge would probably precede the 1:12 scale model testing, thereby eliminating the need for the latter. The full scale testing of the bridge was to be scheduled closely following the rehabilitation of the deck in 1982. Therefore, the new schedule of research and development work was to be centred around the deck replacement.

The main span deck was to undergo a preliminary load testing in the fall of 1981. The replacement of laminated deck and upgrading of the main span floor beams would follow in the spring of 1982, followed by full scale load testing in the summer or fall of 1982.

Subsequently the entire structure would be re-evaluated using the results of the load testing and analysis. This work would aid in identifying any additional structural deficiencies and in developing feasible methods of rehabilitation for application at a future date.

The remaining sections of this report cover the preliminary load testing, deck rehabilitation, full scale testing, and the subsequent re-evaluations, including proposed methods for additional rehabilitation.

7/ PRELIMINARY LOAD TESTING AND ANALYSIS OF THE MAIN SPAN DECK

The main span deck system was to be load tested and re-evaluated prior to the replacement of the laminated deck. This was done to generate a realistic analytical model of the deck system for the evaluation of over-stress conditions in the steel bars of the floor beams.

7.1/ Load Testing

A preliminary load test of the main span deck was conducted in the fall of 1981. Only one test vehicle was employed with its legal complement of 18 loading blocks. Instrumentation was kept at a minimum as MTC's computer-based scanning system was not used.

7.1.1/ Instrumentation

Electrical resistance strain gauges were used to monitor the strains in the steel bars of the king post floor beams. Both bars of the first eight floor beams from the south end of the main span were instrumented as shown in Figure 16. These gauges were monitored individually using a strain balance unit and the values were recorded in tables.

7.1.2/ Vehicle Loading and Placement

Only one test vehicle was used as shown in Figure 17. The limited number of loading blocks were rearranged to produce several different dual axle tandem weights as shown in Table 14. The rear tandem was the heaviest and represented the reference location for vehicle placement.

The vehicle was placed in three transverse positions or lane positions as depicted in Figure 18. These represented the assumed east and west driving lanes as well as a symmetrical position about the centreline of the roadway.

In each of these lanes the vehicle was oriented facing southbound so that its rear tandem would be furthest on to the bridge. Along the west lane and centreline the vehicle was placed so that the rear tandem would be located in three positions over each of the first five floor beams from

the south end, as indicated in Figure 19. These three positions represented each of the two rear tandem axles directly over each floor beam as well as the axles symmetrically placed about the floor beam. For the east lane, only the rear-most axle was positioned over the floor beams.

At each stationary position of the vehicle as described above, for each of the three load levels, all 16 strain gauges were monitored. Zero offset values were recorded before and after the vehicle moved through each lane.

7.1.3/ Testing Results

The results of the preliminary load testing indicated that the structural performance of the floor beams under the test vehicle was reasonably consistent under the different load levels and positions. A representative set of test data is provided in Table 15. This data consists of the average of the steel strains from the two bars on each floor beam for load level three in lane position two. The longitudinal wood stringers in the deck are lapped over the odd numbered floor beams as identified in Figure 16. The maximum measured strains occurred in the odd numbered floor beams where the stringers were lapped. This conflicts with ideal conditions in which the two span continuous stringers would impart maximum reactions to the floor beam in the middle.

Figures 20 through 25 display the steel strains in all the instrumented floor beams under a number of vehicle positions. These figures represent the three axle positions A, AB and B as identified in Figure 19, for floor beams three and four. These plots indicate that very little longitudinal distribution of load occurs beyond these floor beams in the immediate vicinity of the axle loads. Figure 23 also demonstrates that even with an axle directly over an even numbered floor beam, the adjacent floor beam where the stringers are lapped developed higher strains.

Initially, these test results were used to validate the distribution assumptions used in the preliminary evaluation. A floor beam was evaluated for the 280 kN tandem of the test vehicle using the simply supported stringer assumption described in Section 5.3.1 of this report.

Using that approach the predicted, maximum strain in the floor beam due to the 280 kN tandem was 444 $\mu\varepsilon$. This value is remarkably close to the maximum values given in Table 15. Therefore the preliminary evaluation for the OHBDC loads was considered adequate.

7.2/ Structural Analysis

In order to evaluate any proposed reinforcing schemes it was necessary to develop a detailed analytical model capable of predicting the structural response of the main span deck system. Initially a three-dimensional frame analysis [15] was proposed and was eventually performed as will be presented. However, the first model applied was based upon a two-dimensional grid analogy [16].

7.2.1/ Grid Analogy

The main span deck was simulated using a two-dimensional grid as shown in Figure 26, which included the first nine king post floor beams from the south end. The end beam over the concrete pier was supported directly on the pier and had no king post. The king posts were represented by spring supports derived from the flexibility of the floor beam as depicted in Figure 27.

In the grid shown in Figure 26, the longitudinal stringer beams each represented the sum of the flexural and torsional stiffness of two stringers in order to reduce the grid size. The first six floor beams represented the sum of the flexural and torsional stiffnesses of the wood beam portion of the king post floor beam and one quarter of the transverse laminated deck between floor beams. The properties of remaining transverse deck between floor beams was accounted for in the deck beams. The last three floor beams represented the same wood beams plus all of the laminated deck between floor beams. The latter modification was incorporated in order to reduce the grid size and was not expected to seriously affect the responses, as no loads were to be applied in that area.

The grid as defined in Figure 26 allowed rotation in both directions at all supports except for rotation about the longitudinal axis of the

floor beams at their ends. Except as will be defined later, all member-to-member connections transferred both moments and torsion.

7.2.2/ Loading Input

All loads were applied as point loads at the joints of intersecting members. These point loads were derived by distributing the wheel loads to the joints assuming simple supports between members. The magnitude and dimensioning of the loadings simulated test vehicle loadings as described in Section 7.1.2 for lane two and under load level three. However, the load positions over floor beam one and part of two were not considered as these members did not produce the highest strains during testing and it was difficult to simulate the member end fixity over the rigidly supported beam at the pier.

7.2.3/ Forces Output

Only floor beam member forces and reactions were specified for output. The primary consideration which will be presented in this report were the reactions of the spring supports at the centre of the floor beams. These spring reaction forces were to be related to the bar forces in an actual king post floor beam. The spring force reaction would be taken as the force in a king post and adjusted by the relative forces between the king post and bars as determined by the frame analysis from which the spring constant was determined originally. Subsequently this simulated bar force would be converted to strain for comparison with the actual test results.

7.3/ Analytical and Experimental Comparison

A number of the grid analogies were performed, each adopting changes in various member connections and material property assumptions. This approach was taken in order to obtain a reasonable comparison of the analytical results with the experimental results from the load testing.

7.3.1/ GRID1 Analysis

The initial grid analysis followed all of the previously discussed considerations described in Section 7.2. In addition, the stringer beams were assumed to be pinned on either side of the odd numbered floor beams as shown in Figure 28. This simulated the actual lack of moment transfer due to the simple stringer supports over those beams.

GRID1 represents the ideal conditions assuming the participation of all members and supports. It assumes the modulus of elasticity to be 13800 MPa (2000 ksi) for the Douglas Fir members, and 6900 MPa (1000 ksi) for the old nailed deck. This constitutes a 25% increase in the specified OHBDC value for Douglas Fir to account for its high density, and a 25% reduction of the OHBDC value for the SPF deck due to decay and high moisture content [5].

The results of the analysis, in terms of simulated strains for the steel bars of the floor beams, are given in Table 16. Compared to the test results in Table 15 the maximum analytical strains are considerably lower. Comparison of the floor beam strain distribution between the test results and GRID1 are provided in Figures 29 through 34. With the tandem axles positioned over floor beam three, as shown in Figures 29 through 31, the analytical results demonstrate a similar distribution but of a lower magnitude. When the tandem axles are positioned over floor beam four, only Figure 33 demonstrates a similar distribution pattern. The experimental strains in Figures 32 and 34 indicate that the floor beams at which the stringers are discontinuous attract more load, even though the load is closer to the adjacent beams. This may be caused by mechanical performance of the deck system due to both the old deteriorated deck and the lack of hold down at the stringer ends.

In terms of predicting a maximum strain value the GRID1 analysis underestimated the experimental results by up to 23%. It did however indicate that this grid analogy could represent a reasonable model of the deck system with some refinements.

7.3.2/ GRID2 Analysis

Considering the deteriorated condition of the old nailed laminated deck, it was believed that its contribution to the distribution of load was very minimal. Therefore, in the GRID2 analysis the flexural and torsional stiffness of the deck was ignored assuming it provided only vertical shear resistance.

The results of this analysis are given in Table 17. Though the maximum anticipated strain increased, it was only by a few percent. The distribution pattern between floor beams for this analysis was not plotted as the values were very close to those produced by the GRID1 analysis.

7.3.3/ 3-Dimensional Analysis

Prior to continuing with the grid analysis method, a three-dimensional analysis [15] was carried out, in which all members including the steel bars and king posts were represented directly. Normally a direct comparison of experimental and analytical results would confirm the validity of an analytical model such as the grid analogy. However, because the deteriorating deck system may have been behaving mechanically, it was decided to further evaluate the effectiveness of the grid analogy by comparing it to a more refined three-dimensional analysis.

The member-frame arrangement is exactly the same as for the grid analysis shown in Figure 28. The only difference was that the king posts and steel bars were represented as actual members in vertical plane, instead of using the spring support. All other considerations were the same as for the GRID2 analysis.

The results of this analysis are compared to the GRID2 analysis in Table 18. In terms of predicting the steel bar strains, the two analyses were very similar. However, the high cost of using the three-dimensional analysis, considering the number of variables yet to be evaluated, prevented it from being used again. The grid analogy was considered to

be adequate for the purposes of developing an economical analytical model for the deck system.

7.3.4/ GRID3 Analysis

The modulus of elasticity of the wood (E) initially assumed of 13800 MPa was higher than that specified in the OHBDC. Therefore the specified value of 10000 MPa was to be used in the next analysis. This reduction in E would tend to increase the reaction at the spring supports as the deck system becomes more flexible. It also reduces the value of the spring constant slightly, but not significantly, as the relative ratio of the E's between the wood and steel of the king post floor beam plays a role in its determination.

An additional variable was considered in connection with the deterioration of the steel bars themselves. These bars have undergone some rusting over the years which effectively reduces the cross-sectional area, thereby reducing their stiffness and increasing the effective strain. For our purposes a 1 mm (1/32") of material was assumed to be removed all around the 44 mm diameter (1 3/4") bars reducing their cross sectional areas by 7%. Rather than adjusting the measured strains from the field testing, this area reduction was accounted for directly in the determination of the floor beam stiffness thereby affecting the spring constant to be used in the grid analysis.

GRID3 accounted for both of the above adjustments, under the same loading conditions as before. The results are given in Table 19. The floor beam strain distributions are plotted in Figures 29 through 34 along with the previous GRID1 analysis.

GRID3 predicts maximum strains much closer to the experimental values, while still demonstrating the same distribution characteristics as GRID1.

7.3.5/ GRID4 Analysis

It is believed that the real deck system behaved as it did because, due to deterioration and looseness, the deck members had mechanical movement.

Though this could not accurately be simulated by the grid analogy, a realistic interpretation was possible for certain loadings.

When the tandem was positioned over a floor beam where the stringers were discontinuous, it is believed that continuity of the stringers was not achieved. Therefore the GRID4 analysis assumed all the same conditions as GRID3 except that all the stringer beams were provided with member end releases as shown in Figure 35. This exaggerates the condition as it virtually eliminates the distribution of wheel loads to any floor beams beyond the immediate vicinity of the applied loading.

The results of this analysis are given in Table 20. Compared to the test results in Table 15 they are similar only for load positions where the tandems are over the floor beams at which the stringers are discontinuous.

7.3.6/ GRID Combination

It is appears the best approximation of the experimental results is a combination of the GRID3 and GRID4 analogies. For loadings over the even numbered floor beams the GRID3 analogy is more representative as the continuity of stringers plays a role in the performances. For loadings over the odd numbered floor beams the GRID4 analogy is better. Table 21 contains this combination of results for comparison to the test results.

Figure 32 through 34 already display the strain distribution for GRID3 for loads over floor beam four. Figures 36 through 38 display the GRID4 results for loads over floor beam three as compared to the experimental results. The latter load positions also represents the maximum measured strains and so is of greatest importance to this evaluation. Accordingly Figures 36 through 38 indicate the refined grid analogy prediction of maximum stress is very close to the experimental values and therefore represents an ideal model for predicting maximum responses.

7.3.7/ Summary

The reasoning behind this extensive analytical approach is directed towards evaluating methods for upgrading the deficiency in the deck. The results of the GRID3 analysis indicate that if we ensure the deck system

performs according to those ideal conditions, even without a good deck, the maximum strains could be reduced by 8% from the 440 με in GRID4 to 409 με in GRID3. With a new deck properly tied down the maximum strain could be reduced by as much as 17% using the maximum value of 367 με from GRID1 as a comparison value. In the latter analysis the E of the deck was assumed to be very low at 6900 MPa. Assuming a higher value as specified in the OHBDC would improve the distribution somewhat.

Though it is difficult to represent the performance of the existing old deck system using this grid analogy, it was considered to be an effective model to evaluate alternative methods of upgrading the deck system. Also, the need for a new laminated deck was to play an important role in approaching a method for upgrading the deck system.

One line of thought involved the increase of longitudinal load distribution which could help considerably in reducing the overstrain condition of the floor beams. Therefore the new deck was seen as a possible means of increasing the longitudinal stiffness of the deck system.

7.4/ Proposed New Deck Replacement

The new laminated deck was made of prestressed wood using a new system of construction developed by MTC (1, 17). These new prestressed wood decks are stronger, stiffer and more durable than the previous nailed systems. The new deck on the Sioux Narrows bridge was to be oriented longitudinally as opposed to the existing deck which was oriented transversely as shown in Figure 39. In addition, the new deck would be 190 mm deep, representing a full standard size increase over the existing 140 mm deep deck.

The overall objective was to increase the longitudinal stiffness of the deck system in order to improve load distribution and lower the stresses in the floor beams. The previous evaluation in Section 5 of this report indicated the steel bars of the floor beams were overstressed by over 90% according to the OHBDC specifications. Therefore utilizing the analytical model derived previously, the proposed prestressed deck was evaluated to determine its effect on the load distribution.

7.4.1/ GRID Analogy

The same grid arrangement as used previously was maintained for the new analysis, see Figure 26. The new longitudinal deck was to be continuous over all floor beams and extend over all the approach spans. It was to be securely tied down to the stringers and floor beams thereby creating continuity of the deck system. The properties of the prestressed deck were determined according to the new OHBDC specifications [18] which are based upon extensive research and development of the prestressed wood system [17]. The longitudinal and transverse, torsional and flexural stiffness of the deck were apportioned to the stringer floor beam and deck grid members along similar lines as for the previous analyses.

Several adjustments to the previous analytical assumptions were employed. No member end releases were provided for any of the stringer beams, as it was believed that the overlapping stringers in conjunction with securely tied laminated deck would create reasonable continuity in the deck system. The modulus of elasticity of the existing Douglas Fir timbers was assumed to be 13 800 MPa and 9650 MPa for the new wood deck. The deteriorated condition of the steel bars, using a reduced steel cross section, was maintained.

The analysis was performed using the same test vehicle loadings and vehicle positions as used in the previous testing and analysis described in Section 7.2.2.

7.4.2/ GRIDN1 Analysis

Table 22 displays the results of the grid analysis incorporating the proposed new prestressed longitudinally laminated wood deck under the same loads as before. The maximum anticipated strain would now be 297 $\mu\varepsilon$ as compared to the 472 $\mu\varepsilon$ measured during the testing of the existing deck. This would represent a reduction in peak strain in the order of 35% with respect to the test value.

Table 6 provided the results of the original evaluation indicating the factored member resistance of 466 kN for the steel beams was only 52% of the factored OHBDC load effect. Therefore the predicted 35% reduction in

load effect as determined from this analysis, was still not adequate to upgrade the deck system for the full OHBDC loads.

7.4.3/ Proposed Proof Test Load

The GRIDN1 analysis was then used to evaluate the new deck system under the effects of the proposed MTC proof testing vehicles. These vehicles would consist of maximum tandem axles of up to 220 kN (50 000 lb.) each. The testing of the floor beams would constitute one vehicle in each lane simulating the simultaneously application of lanes one and three as described in Figure 18. Under the effects of this proof loading this analysis projected a maximum strain of 680 με. The dead load strain, accounting for the new heavier deck and 75 mm of new asphalt, would be 128 με. Therefore, the total field strain under proof load was projected to be about 808 με. Adjusting this to account for the net section at the thread, the maximum strain would be 1115 με just short of the assumed yield strain of 1140 με.

7.5/ Conclusions and Recommendations

The proposed prestressed wood deck would not improve the longitudinal distribution of load sufficiently to meet the OHBDC requirements. However, alternatively the new deck should improve the deck performance enough to allow the application of the MTC proof test loading. In the past the successful application of the fully loaded MTC test vehicles, weighing up to 900 kN each with up to 450 kN tandems, has been considered acceptable in confirming the adequacy of bridges and bridge components. The basis of this approach being that the application of actual loads in the field confirms both material performance and load response so that the conservative load factors and performance factors in the OHBDC may be adjusted for the member responses being tested.

Because of the results of the analysis and the fact that existing laminated deck needed immediate attention, the installation of the proposed prestressed wood deck was approved. This approval was subject to a structural review of the other components of the deck system which included the stringers, wood beam and king posts all of the which were determined to be adequate. It also required that the completed deck be proof tested to verify its structural capacity.

8/ DECK REPLACEMENT

The new prestressed wood deck, to be designed by MTC, was tendered for construction by normal procedures. Due to the relative newness of the prestressed wood system, site construction was to be monitored closely.

8.1/ Prestressed Deck Details

The design of the new prestressed wood deck was performed according to the new OHBDC specifications [18]. With the existing longitudinal stringers its capacity was far greater then is required structurally.

Detailed structural drawings are available under MTC file Site 41S-71 and Contract 81-226. The basic arrangement is shown in Figure 40. The new longitudinal deck was to be installed on the existing stringers, supported on 20 gauge, 300 mm wide galvanized metal strips. These strips would run perpendicular and on top of the stringers at 2 to 3 m intervals so that laminations wouldn't drop between stringers during construction. The deck was to be prestressed at 1.5 m intervals using 25 mm diameter Dywidag post-tensioning bars in pre-drilled holes at mid-depth.

8.2/ Construction Procedure

The actual deck replacement occurred in the spring and early summer of 1982. In order to maintain traffic flow, the new deck was installed in two halves. A temporary guard rail was placed along the centreline of the bridge and one half of the old deck was removed and replaced with the new longitudinal deck as shown in Figures 40 and 41. Once completed the new half was prestressed, tied down and overlayed with a thin asphalt surface as shown in Figures 40 and 42. The traffic was then rerouted onto the new half, while the second half was being replaced. Once completed, the second half was prestressed against the new half by coupling the prestressing bars of the two sections together along the centreline of the bridge. By stressing from the new side, the anchorage nuts along the centreline at the temporary steel bulkhead disengaged and moved into the new half of the deck. This created a two-lane continuous deck, as shown in Figure 43, which was then asphalted and opened to traffic. The rehabilitation included new wood curbs, guardrail posts, railings and a new outrigger sidewalk as shown in Figure 44.

9/ LOAD TESTING AND RE-EVALUATION OF MAIN SPAN DECK

The completed deck system was load tested once again in the fall of 1982 along with the south approach span and the main span trusses. The approach span and the main span trusses will be discussed in subsequent sections.

The objective of this load testing was to determine the applicability of the GRIDN model developed in Section 7 and also to perform a proof test of the rehabilitated deck.

9.1/ Loading Testing

The first testing included a repeat of all the loads and positions applied during the preliminary testing. Subsequently the deck was load tested to the maximum loads given in Table 23.

9.1.1/ Instrumentation

Instrumentation included the same steel strain gauges on the first eight floor beams at the south end of the main span as shown in Figure 16. However, these gauges were now to be monitored using MTC's data acquisition system, an action which would considerably increase the efficiency of the testing program.

9.1.2/ Vehicle Loading and Placement

The first testing involved only a single vehicle and was a repeat of the loads and positions as described in Section 7.1.2. Subsequently the proof testing of the deck system was performed using the loads given in Table 23. The testing was performed under these incrementing loads in order to ensure the structure maintained linear elastic performance.

The transverse or lane vehicle positions for the proof testing consisted of both testing vehicles one in each lane, as shown in Figure 45. This was achieved by placing a single vehicle in each lane alone, then both vehicles together, creating three lane positions.

The longitudinal positions for the proof testing of the main span deck are the same as given in Figure 19 except only position A was used and the vehicles were facing in the opposite direction. Only one position over each floor beam was performed in order to reduce the time required to complete the testing.

9.2/ Analytical and Experimental Comparison

The results of the testing of the main span deck will be presented in two parts. The first testing designated as TEST 1 involved the repeat of loads and positions used in the preliminary tests. The second testing, designated Test 2, consists of the proof testing using two vehicles.

9.2.1/ Test 1

The results of the testing for lane two under load level three are given in Table 24. In terms of the maximum strains, the grid analysis GRIDN1 in Table 22 compares favourably with these test results. However, the test results indicate that the deck system is still not completely continuous as the strains are still lower for loads over the even numbered floor beams. It was not possible to simulate the semi-continuous nature of the new deck system but the GRIDN analysis was considered adequate to predict maximum strains for the new deck system.

Figures 46 through 48 display the strain distributions for several load positions from the test results before and after the deck was rehabilitated. These plots demonstrate a dramatic reduction in floor beam strains due to the new laminated deck. From Figure 47 the least reduction in peak strain is about 36% which compares well with the predicted value of 35% from the GRIDN1 analysis.

9.2.2/ Test 2

The results of the proof testing of the main span deck under the two vehicles at load level nine are given in Table 25. The peak measured strain was 665 $\mu\varepsilon$ in floor beam five which compares well with the projected maximum value of 680 $\mu\varepsilon$ using the GRIDN1 analysis. Therefore accounting for the strain due to dead load of 128 $\mu\varepsilon$ and the net section

at the thread, the floor beams were proof tested to a strain of 1094 με, just short of the assumed yield of 1140 με.

9.2.3/ Summary

The results of the testing confirmed the structural strength of the new deck system and the ability of the GRIDN1 analysis to predict maximum steel strains. Therefore the analysis was to be used in the re-evaluation of the main span deck system. The new deck system successfully carried the fully loaded MTC proof test load consisting of 445 kN dual axle tandems.

9.3/ Re-evaluation

The main span deck system was re-evaluated for the OHBDC loads using the derived GRIDN2 analogy. The main emphasis was placed upon the transverse floor beams for which only the results for the wood beam and steel beam will be presented. The wood king post, stringers and laminated deck did not display any structural deficiencies.

9.3.1/ Factored Load Effects

Because of the increase in both laminated deck thickness and the asphalt thickness, the factored dead load effects for all components were increased. However, due to the improved load distribution, the live load effects were reduced. The total factored load effects are listed in Table 26 and may be compared with Table 6 from the preliminary evaluation. This comparison shows a marked reduction in load effects with a maximum reduction of nearly 40% in the steel bars.

9.3.2/ Wood Beam

The combined effects of the tabulated moment and axial forces produces a maximum interaction of 0.78. Therefore the wood beam component of the floor beam presented no structural deficiencies.

9.3.3/ Steel Bars

The steel bars, though demonstrating a marked reduction in load effects, still do not meet the requirements of the full OHBDC loading. Although the full net section has been used, the overstress is about 18%.

9.4/ Conclusions and Recommendations

The load testing and analysis of the king post floor beam system verified the applicability of using a grid analogy to represent this structural system. The application of the new longitudinal prestressed deck reduced the overstress condition in the steel by 35%. However, the steel bars are still overstresed when evaluated under the full OHBDC loading. In determining consequences of this condition it is necessary to consider several possibilities.

The restricted width of the structure at about 7.3 m between curbs, and the limited frequency of heavy vehicles may reduce the probability of simultaneous occurrence of two fully loaded heavy vehicles. In addition the lower speed limit at the structure location of 50 km/h also reduces the "probable effects" of dynamics.

As an example, if only one OHBDC vehicle was considered, the total factored load effect in the bars would be 424 kN which would then be within the resistances provided in Table 26. Alternately, if the simultaneous occurrence of two vehicles was equal to 70% of the OHBDC loads, instead of 90%, the total factored load effect would be 450 kN. Both of these considerations would bring the evaluation within acceptable levels.

It is also necessary to consider the results of the proof testing. The two 445 kN tandems of the test vehicles represent 85% of the maximum factored OHBDC loading, where load factor, dynamic allowance multiple presence and the difference in axle spacings are considered. In the past, when the resistance of a structure could not be realistically determined either due to deterioration or geometry, the successful application of the MTC proof loads was considered adequate. Therefore the main span deck system was to remain unchanged.

10/ LOAD TESTING AND RE-EVALUATION OF THE SOUTH APPROACH SPAN

The south approach span was also load tested in the fall of 1982 in order to provide a basis for the development of an analytical model to be used in the evaluation. The orientation of the new deck parallel to the existing stringers complicated the utilization of the simplified analytical methods given in the OHBDC. The load test results were to be used to develop a grid analogy capable of predicting the maximum responses under design loads.

10.1/ Load Testing

The load testing of the south approach span was conducted in conjunction with the proof loading of the main span deck. Therefore the first three load levels, which were to simulate the preliminary testing done in 1981 on the main span, were not applicable.

10.1.1/ Instrumentation

The instrumentation included displacement transducers at the midspan of each stringers as shown in Figure 49, to monitor deflections. In addition, six relatively new demountable strain gauges were applied to the six outside stringers on the west side as shown in Figure 49. The latter were applied to the bottom faces of each stringer to monitor extreme fibre bending strains.

10.1.2/ Vehicle Loading and Placement

The transverse or lane positions of the vehicles and the load levels were identical to those applied for the proof testing of the main span deck as described in Section 9.1.2. The longitudinal vehicle placement is shown in Figure 50, and this position was applied for all lane positions for load levels four through nine.

10.1.3/ Test Results

The range of the linear displacement transducers used in the south approach span were only 50 mm. Following standard procedures they were

installed with available movement in both the plus and minus directions. This drastically reduced the measurable movement that could be monitored during testing. Consequently the range of these gauges was exceeded on many occasions as will be discussed in the subsequent section.

10.1.3.1/ Deflections

Figure 51 displays the midspan deflections results for increasing load levels for two lanes loaded as shown. Except in certain instances many gauges displayed apparent non linear action above load level four. This was caused by the actual deflections exceeding the available range of the gauges. A number of gauges reached higher values only because they were readjusted during the test in order to provide several maximum deflections under maximum load. It was not possible to readjust all the gauges due to limited access and imposed time restrictions. As a result only the deflection output for load level four will be used for comparison with the analytical model to be derived later.

Figure 52 displays the load versus deflection results for stringers three and five. These gauges were adjusted to measure the full range of movement. The results indicate that the structure responded linear elastically.

10.1.3.2/ Test Results Strains

Only the six outside stringers shown in Figure 49 were monitored for strain on the bottom faces. Figure 53 displays the measured strains for all load levels under the load position shown. The results demonstrate a reasonably consistent distribution pattern under increasing load.

Figure 54 displays some of the results in terms of load versus strain. It is apparent that some transition of response occurs somewhere between load levels four and five, beyond which the strain are reasonably linear. It is believed that this may be the result of overcoming the resistance to relative movement between the new deck and the existing stringers. However, the strain gauges did not display the residual measurements after testing which usually would occur if the structure were permanently set.

These demountable strain gauges were relatively new at the time of application. In addition, wood strains are rarely monitored due to a variety of factors. The measurement of wood strains is believed to be very sensitive to numerous variables such as modulus of elasticity, slope of grain, and vehicle placement [19]. Though this strain data is given for comparative purposes, it should be used with caution.

10.2/ Structural Analysis

The south approach span was re-analysed after testing using a grid analogy [16]. The grid arrangement is given in Figure 55.

Each longitudinal member represented one sawn wood stringer and the portion of longitudinal deck equidistant between adjacent stringers. The torsional and flexural properties of the longitudinal members were derived by summing the individual properties of the stringer and deck, as determined using the idealizations given in the OHBDC.

Each transverse member represented a transverse width of deck of about 910 mm. The flexural and torsional properties of these member were also derived using the idealization given in the OHBDC.

The properties of the longitudinal members on the outsides and the one along the centreline of the span were adjusted to reflect the added stiffnesses provided by the steel channels used for prestressing as shown in Figure 40.

The continuity of the new deck beyond the north end of the span was initially represented by extending the longitudinal members one panel beyond the support. The properties of these extended members reflected the deck only.

All member-to-member connections were assumed to be rigid. Initially the moduli of elasticities were assumed to be 11000 MPa for the stringers and 9650 MPa for the decking.

10.3/ Analytical and Experimental Comparisons

The above grid was analysed for loads simulating the effects of load level four for the two vehicle loading shown in Figure 45, as well as for the individual vehicles placed separately in each lane.

10.3.1/ GRIDS1 Analysis

The results of the GRIDS1 analysis as compared to the experimental data given for vertical displacements at midspan are shown in Figures 56 through 58. Figure 59 displays strain comparisons in which the GRIDS1 results represent the member moments apportioned to the stringer sections according to the assumed relative stiffness between the stringer and deck components.

In all instances the GRIDS1 analysis tends to underestimate the measured values. It is important to realize that due to the narrow stringer spacing the output is extremely sensitive to the vehicle position. In the field an acceptable transverse vehicle placement would be within 150 to 200 mm.

10.3.2/ GRIDS2 Analysis

The GRIDS2 analysis involved several changes to the grid analogy. The extension of the longitudinal deck members beyond the north support was increased to 2290 mm to reflect the actual floor beam spacing in the main span. The relative stiffness of the stringers was increased to reflect a modulus of elasticity of 13790 MPa instead of 11000 MPa.

The results of the GRIDS2 analysis are much closer to the experimental values for both deflections and strains. However, the analysis appears to provide a better transverse distribution of load with lower relative peak values than provided by the testing.

10.3.3/ Summary

The analysis assumes the new deck is fully tied down, with continuity in all directions with respect to the stringers. No composite action is

assumed in the member properties. In the field, the deck is not perfectly tied down and so it does not provide full continuity. However, considering the range of variables involved, such as relative modulus of elasticity, vehicle placement, and member-to-member connection stiffnesses, the analysis results are reasonable.

It is not practical to further modify the analysis by randomly adjusting the member properties without sound reason. Therefore the GRIDS2 analysis was to be used in the re-evaluation of the span for the OHBDC loads. In addition, comparative load distribution criteria was to be derived using the simplified methods in the OHBDC as well as directly from the test results.

10.4/ Re-evaluation

The south approach span was re-evaluated using three methods of determining load distribution as will be described. Though both the deck and stringers were evaluated, only the stringers will be presented here as the deck did not display any deficiencies. Member resistances for the stringers are as derived in the preliminary evaluation given in Table 4.

10.4.1/ Factored Load Effects OHBDC

Initially the evaluation was performed according to the OHBDC using the simplified method of analysis as presented in Section 3 of the Code. As with the previous grids analysis, the torsional and flexural properties of the new bridge were assumed to be equal to the sum of the properties of the individual deck and stringer components derived according to the OHBDC idealizations. The derived distribution widths are given as compared to those derived during the preliminary analysis in Table 27.

With the longitudinal re-orientation of the deck the distribution width for moment has been reduced. However, following the OHBDC specifications this re-orientation has no effect on the shear distribution width.

Using these distribution widths, as in the preliminary evaluation, the total factored load effects as compared to the factored resistances are given in Table 28. The span was assumed to be simply supported even

through some continuity is provided by the new deck. The portion of the total factored moment effect assumed to be carried by the stringer was determined according to the relative stiffness between deck and stringers. Initially the stringer was assumed to carry all of the shear force.

According to this evaluation the shear capacity was considered to be adequate. Even if there were a small contribution from the new laminated deck, the basic OHBDC shear resistance would be larger than the load effects. Accounting for the superior quality of the stringers would also make the shear resistance much higher. Therefore the shear resistance was not further considered.

The moment resistance, however, even when taking the higher material quality in to account, falls short of the factored load effect. Therefore, the structure was analysed using the GRIDS2 analysis model in order to determine a more realistic distribution.

10.4.2/ Load Distribution Using GRIDS2

The structure was analysed for the factored OHBDC design truck with two lanes loaded. All applicable modifications factors were included. The results provided a maximum live load moment of 132 kN-m/member: the member included both the stringer and deck components. Summing all of the moments across the full width of the bridge provide a total moment of 1417 kN-m. Adjusting this total to remove the modifications for the loading of two lanes provides a maximum moment of 787 kN-m/lane. Therefore, the 132 kN-m/member maximum moment represents a distribution fraction (S/D_d) of 0.335, for one line of wheels, and provides an equivalent distribution width D_d of 1.36 m which now includes the effects of the modification for two lanes loaded. This is even more critical than that derived using the OHBDC simplified method as given in Table 27. However, the previous application of the simplified method assumed a simply supported span, thereby creating larger initial maximum moments. The undistributed factored maximum OHBDC moment for two lanes loaded is 1785 kN-m as compared to the 1417 kN-m derived from the GRIDS2 analysis which includes the effects of span continuity. Therefore, using the grid analogy the maximum moment is 80% of the simply supported moment

while the D_d value is 89% of that determined using the simplified methods. These opposing changes provide a new maximum total factored moment of 157 kN-m/member of which 116 kN-m is assumed to be carried by the stringer as shown in Table 29. This latter value is now based upon the relative stiffness of the deck and stringer components and the continuity of the deck. The 116 kN-m is still larger than the OHBDC resistance of 86 kN-m but is closer to the value based upon the adjusted specified strength.

10.4.3/ Load Distribution Using Test Deflections

A final comparison was made between the GRIDS2 model and the test results in efforts to verify the validity of the model to predict load distribution.

In general terms, where uniformly spaced prismatic members are involved, the relative magnitude of the member deflection closely reflects the distribution of the member moments. Therefore the actual maximum measured test deflections, relative to the sum of all the measured deflections across the structure can be used to approximate the load distribution characteristics of the bridge under a given loading. Under load level four, with both test vehicles on the bridge, the maximum deflection was 29.1 mm. The sum of all the deflections across the width of the bridge under the same loading was about 419 mm. Treating these numbers the same as for the previous moments we obtain a simulated distribution width D_d of 1.82 m. This value cannot be directly compared to the numbers derived using the OHBDC loads as the axle spacing and vehicle positions were quite different. However, using the GRIDS2 model under the same test loading the maximum deflection was 27.7 mm with a total across the width of 420 mm. This provides a simulated distribution width of 1.92 m. Alternatively, using the moments from the same GRIDS model analysis the simulated D_d would be 1.82 m. Therefore this indicates the GRIDS2 model is probably providing distributions within 5% of the values given by the test results.

10.5/ Conclusions and Recommendations

The load testing and analysis of the south approach span verified the applicability of using a grid analogy to represent this structural system. The re-orientation of the new laminated deck reduced the effective lateral load distribution but at the same time improved, by a greater margin, the factored resistances. The new structural system is considered to be adequate to carry the full OHBDC design loads with certain considerations.

The OHBDC horizontal shear resistances as given in Table 28 were determined using only the stringer section. Any one of three applicable adjustments would result in a satisfactory shear design condition. Either including the shear resistance of the new deck, adjusting the specified strength or assuming reduced end splits would provide an adequate shear resistance.

The total factored OHBDC moment resistance, as shown in Table 29, exceeds the factored OHBDC load effects. However, the individual stringer resistance is 26% short of the apportioned OHBDC factored moment assumed to be carried by the stringers. The relative stiffness of the stringer and deck components are based upon moduli of elasticities of 9650 MPa and 13 790 MPa. It is likely that such a large difference does not actually exist and adjustment would reduce the apportioned moment assumed to be carried by the stringer. The factored moment resistance of the stringers based upon the adjusted specified strengths is only 6% short of the factored load effects. It is recommended that no deficiency in moment resistance be considered for the south approach span.

11/ LOAD TESTING AND RE-EVALUATION OF MAIN SPAN TRUSSES

The main span trusses were load tested in the fall of 1982 at the same time as the main span deck and south approach span. The object of the testing was to evaluate the effects of the loose secondary members and subsequently derive an analytical model to be used in the re-evaluation of the trusses.

11.1/ Load Testing

The actual testing of the main trusses was conducted as a completely separate test procedure from the main span deck and south approach span. The data acquisition system which monitors the instrumentation was limited to 100 channels and the main trusses themselves were to have nearly 80 channels. In addition, the critical loadings for this testing would be related to gross vehicle weight as opposed to individual axles or tandems as was the case when testing for local deck responses.

11.1.1/ Instrumentation

The majority of instrumentation was applied to the west truss with a limited number of gauges on the east truss for comparative purposes. The overall instrumentation layout is displayed in Figure 60.

The vertical steel bars on the trusses were monitored for steel strains as shown. A gauge was placed on each of the three bars, at the site indicated, and each gauge was monitored separately.

Several primary wood members in the west truss were also monitored for strain using the demountable strain gauges shown in Figure 61. The main top, bottom and end chords which are made up of four laminates were instrumented as shown in Figure 62. Each laminate was instrumented on the top and bottom providing eight readings at each member location. The internal primary diagonals are made up of three laminates with the centre one discontinuous at midlength where the opposing secondary member passes through. Gauges were applied to the top and bottom of the two main laminates as shown in Figure 62.

11.1.2/ Vehicle Loading and Placement

The vehicle loadings for the testing of the main truss span were derived to produce increasing gross vehicle weights with reasonably equal tandems. The seven load levels used are displayed in Table 30. The average maximum 885 kN (200,000 lb.) vehicle is the physical limit of the specially designed vehicles and represents over two and one half times the vehicles legal load.

The same transverse or lane positions as before were used with a vehicle placed individually in each driving lane and then the two placed simultaneously as shown in Figure 45. In addition, a fourth position was used where the respective vehicles were placed along the centreline of the roadway in tandem.

Six longitudinal positions of the test vehicles were employed as shown in Figure 63. These positions represented maximum member responses as derived using the influence lines generated in Section 5. Positions one to five were employed for all four lanes positions described previously. Position six could not be employed for the two lanes loaded as both vehicles were used in tandem.

11.1.3/ Test Results

The load testing proceeded as planned up to load level four during which time the results were monitored consistently in order to project anticipated strains in the next load level. As predicted, the vertical steel bars displayed very high strains due to the non-participation of the secondary members. Figures 64 and 65 display the average strain results for load level four with the vehicles in longitudinal position three (see Figure 63). Each value represents the average result of the all gauges employed at that location. This load position represents the most critical employed during the testing.

Table 31 summarizes these strain results from the two vehicle position as taken from Figures 64 and 65, and also includes the respective dead load strains as determined using the frame analysis from Section 5 assuming no secondary members. The last two columns compare the total assumed member

strains under load level four as compared to the assumed ultimate strains. This comparison indicated that at load level four we were within 15% of the assumed yield strain in some of the steel bars while the wood members displayed more than adequate reserve. At this point it was confirmed that vertical steel bars were the most critical truss members.

Table 32 displays the assumed dead load forces for the vertical steel bars in terms of member stresses and subsequently identifies the stress ranges available for live load accounting for the net section at the thread. Therefore the actual measured live load stress should not exceed the values in column three of Table 32. The fourth column provides the actual peak measured stresses from load level four for each member considering each strain gauge independently. Subsequently the last column projects by linear extrapolation the same steel stresses for load level five. Accordingly, under load level five, some of the members would exceed the assumed yield strength.

As a result of this on-site monitoring, the loadings with two vehicles were discontinued after load level four. The testing was continued on to load level five only to provide the necessary data to perform overall linearity checks. Figure 66 shows several member responses as strain versus total applied load for the vehicle loadings in the west lane. Considering the limited accuracy of the vehicle placement the results indicate a reasonably linear elastic response.

11.2/ Structural Analysis

The main truss was re-analysed using the frame analysis model given in Section 5 and displayed in Figure 13, assuming both participation and non-participation of the secondary members. The analysis was performed simulating load level four for the first three lane positions described in Section 11.1.2 and longitudinal position three as given in Figure 63.

The wheel loads of the test vehicles were evaluated using the grid analogy derived in Section 9.0 in order to provide a more realistic distribution of loads to the floor beam reactions. Subsequently these floor beam reactions were applied in the frame analysis of the truss as

member loads on the bottom chord. The modulus of elasticity of the wood members was assumed to be 13 790 MPa.

11.3/ Analytical and Experimental Comparison

Figures 67 through 72 give a comparison of member strains of the experimental and analytical results for the test load positions described in the previous section. Considering the possible variations in load positions, modulus of elasticity, member looseness and gauge accuracy, the comparison between the first two sets of values is remarkably good.

In nearly every instance the experimental strains fall between the two analytical models and, in each case, the value was much closer to the model in which secondary member performance was not a factor. These results indicate that the latter analytical model is an excellent representation of the structural responses of these trusses. If anything, this model provides conservative results predicting slightly higher stresses than actually exist.

Therefore, the analytical model which ignored secondary member performance was to be used for the re-evaluation of the trusses for the OHBDC design loads.

11.4/ Re-evaluation

The analytical model selected in the previous section for use in the evaluation was identical to that which was assumed for the preliminary evaluation used in Section 5. Therefore the original evaluation would still stand as given in Section 5.4.

The evaluation indicated that, under the full OHBDC design load, a number of members were understrength even when assuming a higher specified strength for the wood. As with the other re-evaluations adjustments to the loading assumptions were considered. First, only one OHBDC truck was considered while both UDL lane loads were retained. Subsequently the load factor for dead load of wood components was assumed to be 1.0 due to the use of actual field measurements. Table 33 provides the approximate reductions in total factored load effects that were derived using these

new load conditions for critical members. These reductions are then compared to the original overload condition derived from the preliminary evaluation. Table 33 indicates that even with the combined reductions not all the members could benefit sufficiently.

11.5/ Conclusions and Recommendations

The load testing and analysis of the main span trusses verified the applicability of the frame analysis, ignoring the secondary members, to represent the structure. However, even taking higher specified strength for the existing wood members, lower dead loads and reduced live loads into consideration, the main trusses display inadequate resistance.

It is recommended that reinforcement of the trusses be considered specifically with respect to the vertical steel bars and horizontal shear at the ends of the bottom chord. The remaining wood members, with the exception of the bottom chord, may be considered adequate based upon their superior quality. However, all cross bracing and framing should be tightened and the secondary truss members should shimmed to engage their ends and so provide lateral support to the primary diagonals.

The bottom chords of the trusses still displayed an overstress of 10%, as given in Table 33, under the full OHBDC loads, even considering the adjusted specified strengths. However, the analytical versus experimental comparison given in Figures 67 to 72 indicates the analytical model used in the evaluation overestimated the actual stress, particularly under heavier loadings. Review of some of the other test results confirmed that, with a test vehicle close to the truss, results for the bottom chord were always below the predicted values. It is possible part of the bridge deck is participating with the bottom chord in tension when under heavy loading near that truss. With this in mind, and considering the difficulties and expense of reinforcing the bottom chord, it was not presented in the final recommendations as a necessity. However, it should be be inspected regularly, and the annual clearing of debris each spring should be maintained.

12/ CONCLUSIONS AND RECOMMENDATIONS

These conclusions are a summary of the inspection, load testing and evaluation results as they relate to overall structural performance. As well, areas of structural deficiency and the methods by which these are determined are identified.

12.1/ Materials

The original site inspections and the subsequent field work has confirmed that the existing structural wood members in the Sioux Narrows bridge are in excellent condition. The theory that the dense select structural wood was of superior quality was substantiated somewhat during load testing in which some strains were up to 1200 $\mu\varepsilon$, which is equivalent to 16.5 MPa. The latter assumes a modulus of elasticity of 13 790 MPa, confirmed by the various analytical and experimental comparisons.

The bar sizes used in the analysis and evaluation were from field measurements and no visible signs of extensive local section deterioration were observed. However, all the steel bar members display a heavy, even layer of scaled rust and the more sensitive threaded areas at the ends of most bars are not accessible. Therefore the calculated capacities of the existing steel bar members should be regarded as questionable.

12.2/ Sawn Wood Stringers and Prestressed Wood Deck

The sawn wood stringer and new laminated deck combination in the south approach span was evaluated in detail. This span represented the most critical of all the wood stringer systems as determined in the preliminary evaluation.

The analysis and load testing confirmed the applicability of the grid analogy to represent this structural system and indicated the existing Douglas Fir members have a relatively high modulus of elasticity. The conclusion of the structural evaluation is that the south approach span is capable of carrying full highway loadings, according to the OHBDC specifications, providing the existing materials are of superior quality.

It is recommended that all the stringer/deck systems in the Sioux Narrows bridge be considered adequate to carry full highway loads without further rehabilitation.

12.3/ King Post Floor Beams, Main Span

The king post floor beam system in the main span was load tested and evaluated before and after the rehabilitation of the laminated deck. These tests confirmed that the grid analogy was an applicable form of analysis for studying this system.

The conclusion of the evaluation is that the capacity of the king post floor beams is governed by the trussed steel bars and all other components of the floor beams satisfy the OHBDC specifications. The new longitudinally laminated prestressed wood deck reduced the stresses in the floor beam bars by about 35%. However, the final structural evaluation indicates that these bars are still overstressed by about 27% according to the OHBDC specifications. The evaluation accounted for moderate adjustment of the bar capacity due to deterioration.

Contrary to the evaluation which suggests that the floor beam bars were understrength, eight of the floor beams at the south end of the structure were proof tested to an equivalent stress level of about 220 MPa at the net section. This is about 95% of the assumed yield and was induced by static loads equivalent to about 85% of the factored OHBDC loads including dynamic load allowance.

Concerns exist regarding the dependability of the bars because the critical thread sections are hidden to view in the wood beam member. In addition, only eight of the 28 floor beams were proof tested. It is possible that the steel is of a higher strength than was assumed in the evaluation but a reasonable sample of full size specimens would have to be tested for this to be confirmed.

Although the king post floor beam system is in no immediate danger under normal highway loads, in light of the above it is recommended that several steps be taken towards ensuring the quality of the bars.

The critical threaded ends of the beams located behind the anchorage nuts as illustrated in Figure 73 should be inspected. If possible, a number of bars should be replaced and the material tested, to determine whether the grade of steel is higher than that assumed in the evaluation. A method of inspection and bar replacement as well as a recommended reinforcing scheme are provided in Section 13.

12.4/ Main Span Trusses

The main span trusses were load tested after the deck was rehabilitated. The testing and analysis confirmed that the assumed frame analogy was applicable, excluding secondary member performance, for the representation of structural response of these trusses. The experimental and analytical comparisons also indicated that the existing Douglas Fir material has a relatively high modulus of elasticity.

It is recommended that the superior quality of the existing dense select structural material be considered in the evaluation as outlined in Section 5.

12.4.1/ Bottom Chord Horizontal Shear

It is difficult to predict the horizontal shear strength of wood as it depends upon the probable shear plane. The horizontal shear in the end of the bottom chords of the trusses is induced by the transfer of the axial compressive forces in the end diagonal through the steel keyways as shown in Figure 74. Assuming an ideal shear plane, as shown, the member was determined to be overstressed by up to 80% for horizontal shear using the OHBDC design loads.

Even accounting for the superior quality of the material, there is still a deficiency. It is recommended that some reinforcement be implemented as soon as possible.

12.4.2/ Wood Members

Other than than the previously discussed horizontal shear, the only significant deficiency in the wood members is the tension in the bottom chord. The bottom chord member is understrength by about 10% using the CSA-086 specifications for member resistance and the OHBDC design loads. This also reflects the superior quality of the existing Douglas Fir material.

The interior compression diagonals were determined to be unstable if the secondary diagonals were not assumed to provide support at mid-length against buckling. In addition, much of the cross-bracing and framing was observed to be loose during the field inspection.

Though the bottom chord in tension does not represent an immediate problem, its failure would more than likely constitute total collapse, as this is not a multi-load path structure, and the trusses are not composed of the fixed member connections commonly found in steel construction. The decision not to proceed with reinforcement of the bottom chord was finalized at a meeting held by the author with Mr. A. Radkowski and Mr. O. Ramakko of the Toronto and Thunder Bay structural offices respectively. Minutes of this meeting, held in Toronto on June 11, 1986, are included in the Appendix.

The instability in the interior primary diagonals caused by the loose secondaries should be rectified as soon as possible. This condition creates severe instability for the very shallow depth members near the centre of the bridge. This can be performed simply by shimming the ends of the members, however, steps should be taken to ensure these shims do not become dislodged with time. All the cross bracing and framing should be tightened, paying particular attention to those areas outlined in Reference 5.

12.4.3/ Vertical Steel Tension Members

All of the vertical steel bars in the main trusses were determined to be understrength by varying degrees as indicated in Table 33.

The evaluation for these members did not take into account any section reduction due to deterioration. Along with this, the detail at the bottom of the bars, as shown in Figure 75, is ideal for water collection, likely causing advanced deterioration at the thread.

Thus it was recommended that steps be taken to inspect some of these members as soon as possible and obtain some material for testing. Subsequently, if this investigation did not improve the outlook for these members, then they would eventually be replaced. A recommended replacement scheme is described in Section 13.

12.5/ Summary

The following summarizes the items which display structural deficiencies in the Sioux Narrows bridge:

1/ horizontal shear capacity of bottom chord in trusses;
2/ axial capacity of vertical steel bars in trusses;
3/ axial capacity of bottom chord in trusses;
4/ secondary diagonals (shimming) and cross bracing in trusses (tightening); and
5/ king post floor beams in main span.

However, only 1 and 4 are at this point being recommended. Both 2 and 5 are to be re-evaluated after field inspection and possible material testing. The axial capacity of the bottom chord was to be considered adequate for the conditions of loading expected on the bridge at present.

13/ PROPOSED INSPECTION REINFORCEMENT AND REPLACEMENT SCHEMES

The following section describes alternative reinforcement or replacement schemes for some of the identified deficiencies. In some instances a recommended replacement procedure is provided for which it is necessary to maintain adequate load carrying capacity during repairs. In some, as well, the methods include recommended member sizes which were derived from the results of the evaluations.

13.1/ King Post Floor Beam Bars

Inspection, removal and possible replacement of bars could be carried out using a simple hydraulic apparatus as portrayed in Figure 76. The lock nuts must first be removed so the extension attachments can be installed. When the bars are pulled by the hydraulic jack the nuts disengage the plate so that they can be moved back to expose the bars behind for inspection. A similar system was successfully applied to the Sydney Creek bridge on Highway 65 in 1976 as shown in Figure 77. Here the vertical bars of the truss were being tightened. The capacity of the jacking system required, with no live load on the bridge, should be at least 50 kN (5 T) for each bar. With live load on the bridge the capacity of the jacks should be at least 150 kN or 17 T. Note that the theoretical capacity of the bar at the net section accounting for deterioration is about 200 kN or 25 T. This value should never be exceeded. While jacking under dead load only, the nuts should disengage the anchorage plate at about 40 kN. Jacking should be stopped as soon as the nut is loose.

If the bars are to be replaced it is recommended that no heavy commercial vehicle be allowed to pass over a floor beam during the time when only one bar is supporting a floor beam. The maximum axle load should be restricted to one half the legal load and the vehicle speed kept to below 30 km/h. The bars should be replaced with 400 MPa (60 grade) steel bars with minimum net section of 1000 mm^2. A recommended material would be 35 M (1 3/8"diameter) Dywidag thread bar reinforcing steel. As shown in Figure 78 the bars could be replaced in two sections using the standard thread bar couplers.

An alternative reinforcing scheme is shown in Figure 79 where an additional steel bar is added. Here vehicle usage is not restricted on the bridge. Detailing of the bearing areas at the bottom of the king post for the new steel anchors block would be necessary to assume an adequate bearing was obtained. New holes would be drilled between the existing bars at the ends of the floor beams. One 35 M Dywidag threadbar of 400 MPa (60 grade) steel is recommended. The 15° offset anchorage holes in the steel anchor blocks allow straight sections of the bar to be used on each side. The new bar should be stressed, under dead load only, to a force of 25 kN.

13.2/ Main Span Trusses

This section deals specifically with the main span trusses.

13.2.1/ Horizontal Shear Capacity of Bottom Chord

It is difficult to reinforce the end of the bottom chord for horizontal shear alone. Normally this strength can be greatly improved by clamps which increase friction along the potential shear plane as shown in Figure 80. However, in this case it is difficult to install such clamps as their placement is inhibited by the bearing block under the chord as can be seen in the figure. Though one cannot predict what the increase in shear strength would be, since the area does not display any problems at present, such a clamping system would probably be more than adequate.

An alternate reinforcement is displayed in Figure 81. Considering the performance and strength of the existing shear key assembly, this reinforcement would be the best approach. Though it would be difficult to ensure a shear force is shared between the old and new shear keys, the latter would at least provide a back-up facility if the existing connection failed.

13.2.2/ Axial Capacity of Vertical Steel Bars

It was recommended that all the vertical steel bars in the trusses be replaced with new, higher strength bars. However, initially only four bars will be removed at midspan to allow inspection and testing of the

material. Subsequently, the remaining bars will be replaced if the investigation shows that they are inadequate. This replacement should be done carefully, as the structure's dead load constitutes the greater portion of the forces in some of these members. Therefore, when a bar is removed, no live load should be allowed on the bridge. As well, care should be taken to ensure the remaining bars share the dead load equally. If they don't, they should be jacked up to equal the loading. A schematic for a proposed jacking arrangement in which each bar is jacked independently is shown in Figure 82.

Replacement bar sizes for all bars are given in Table 34 assuming the use of Dywidag threadbar with a minimum yield strength of 400 MPa. The theoretical member dead load forces are also given. These should only be used as indicators of the force level at which the nuts should disengage the steel bearing plates. The force per bar should not exceed the maximum jacking values listed in the table and jacking should not continue beyond the point at which the nuts are disengaged. The last column in the table provides the total bar elongation under the maximum jacking force, which indicates that only very small movements will take place during jacking. These elongations are necessary so that enough thread above the nuts is made available so the nuts can be moved back in order to release the bar later.

The lock nuts should be loosened and removed only when that set of members is ready for replacement. All three bars should be jacked together using individual jacks with a common hydraulic source. Each should be protected with an in-line one-way valve to to allow independent operation and to prevent sudden unloading of the jacks due to hydraulic failure.

When the three old bars are jacked up to disengage the nuts, the total jacking force should be recorded. The centre bar should be prepared for release by moving the disengaged nut back at least the distance equivalent to theoretical elongation tabulated in Table 34. With the other two jacks isolated, the load on the bar should be released slowly and the bar removed and replaced with the specified new one. The new bar should be jacked up so that the total jacking force on all three jacks equally

sharing the load is again equal to the originally recorded value. The new anchorage nut should then be turned tight against the plate with a wrench, since Dywidag threadbars display considerable anchorage slip under load transfer as compared to machine threads.

If necessary, all bars should be replaced in this manner making sure the sizes, grades and forces specified are maintained. It is recommended that an independent check of the specified member forces and sizes given in Tables 34 be performed prior to commencing any actual work.

13.2.3/ Secondary Diagonals and Cross Bracing

All of the secondary diagonal members should be shimmed at their ends to ensure that they provide the necessary support against buckling of the primary diagonals. Driving wood shims is not recommended, as they will eventally work loose under the effects of normal loading. It is recommended that steel shims be used, which can be tack welded to the steel thrust blocks so that they stay in place.

A large number of the steel bar cross bracing above and below the roadway were noted to be loose during the previous inspections of the bridge. It is recommended that all of these bars be tightened during any future work on the bridge.

13.3/ Summary

The preceding discussions on reinforcement and member replacement are intended as a guide for developing a practical rehabilitation program. Where member forces and/or member sizes are specified they are derived from the extensive analytical work performed on the structure.

Though all deficiencies have been discussed, it should be noted that the most serious problem requiring immediate attention involves the vertical steel bars and horizontal shear in the bottom chords of the trusses.

14/ REFERENCES

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15/ APPENDIX

MINUTES OF MEETING

Subject: Sioux Narrows Bridge Rehabilitation
Hwy. 71, Site 41S-71, District 20, Kenora
W.P. 297-85-01

Date: June 11, 1986 - 1:30 p.m.

Place: 4th Floor Boardroom, 3501 Dufferin Street, Downsview

Present: Ray Taylor - Structural Research
August Radkowski - Structural Office
O.E. Ramakko - Structural Section N.W. Region

Purpose: To discuss the repairs to be carried out to the structure under W.P. 297-85-01 in 1987

Discussion

1. O. Ramakko noted that the following list of work has already been added to the program:

   a) Extension of sidewalks at the approaches to the bridge. (W.P. 297-85-02; C.C. 4381)
   b) Upgrading of the guiderail system across the bridge and approaches. (W.P. 297-85-03; C.C. 4381)
   c) Shimming of diagonal truss members that are no longer in full contact at each end. (W.P. 297-85-03; C.C. 4381)

2. O. Rammako noted four additional items that are identified in the latest Research Report prepared by R. Taylor. These items include:

   a) Provide reinforcement for the bottom chord of the main span truss.
   b) Provide reinforcement for the transverse floorbeams in the main span truss.

c) Replace the steel hangar rods in the main span truss.

d) Reinforce connections between the end diagonal members and the bottom chord in the main span truss.

R. Taylor reported that items a) and b) above have been discussed at length with various head office staff and it has been concluded that reinforcement of these members will not be necessary. These members are apparently overstressed to some extent under full OHBDC loading, but the overstress is marginal and within a reasonable tolerance.

R. Taylor confirmed that items c) and d) noted above are of serious concern to him and he urged that some action be taken to rectify the structural deficiencies in the affected members.

3. O. Ramakko and A. Radkowski outlined their reservations over the proposed repairs required for items c) and d). Specifically these reservations are:

Item c) - Replace all Steel Hangar Rods

i) Based on a recent investigation there is no evidence of corrosion or distress in these steel hangars or in any other main span truss components. It was observed that these members have been performing satisfactorily for 50 years and hence it is difficult to accept that their replacement is an urgent priority.

ii) A Regional estimate for the replacement of all the steel hangars is approximately $250,000. This estimate includes a substantial amount for "Access to Work". This appears to be a very high cost to rectify a deficiency which has not yet and perhaps may never occur.

iii) There is a concern that the new steel hangar bars installed in this bridge may corrode or deteriorate at a much higher rate than those bars presently in the bridge. It was noted that the original hangars show remarkably little deterioration, despite their age. To ensure that new bars have similar non-corrosive characteristics, they may have to be specially made from high-tech alloys.

iv) There is some doubt as to the actual strength of the existing steel hangar rods. This information has not been obtained definitively. The Research Report by R. Taylor indicates that if the rods have a yield strength higher than that assumed, there may in fact be no structural deficiency and hence no reason to replace the rods.

v) There is concern over the procedures and equipment required to successfully replace all of the steel hanger rods in this bridge. While hangar rods have been replaced in other timber truss bridges, none have had the size and complexity of this structure. The concern is that, any procedure detailed by the Ministry may not work. This may result in an expensive redesign following award of a contract. In addition, concern was expressed that the process of installing and stressing new steel hangar rods may, due to the inaccuracies of construction techniques, result in an undesirable redistribution of load and forces in the truss.

As a result of these concerns, it was agreed that the most appropriate approach would be as follows:

- Remove only four of the smallest diameter rods from the bridge as part of the Rehabilitation Contract. These rods would come from four separate locations.

- The bars will be examined and tested to establish yield strength.
  - If the actual yield strengths determined are as assumed, a future contract would be arranged to replace all bars.
  - The replacement of only four bars would permit us to monitor the replacement procedures and equipment required and make the adjustments for a future contract.

Item d) - Reinforce End Diagonal Connections

Based on a recent investigation, there is no evidence of distress in the connections between the end diagonal members and the bottom chord in the main span. While acknowledging that theoretical analysis may indicate a potential overstress in horizontal shear in these connections, it should be noted again that 50 years of satisfactory performance is difficult to ignore.

It was noted that similar connections in other timber truss bridges have failed in the past. This failure generally has resulted from decay in the bottom chord member at these locations.

It was noted that reinforcement of these connections would not be a very expensive item. R. Taylor observed that a failure of this connection could have disastrous results and he strongly urged that some reinforcement be added. It was agreed therefore to add this work to the project.

4. The structural office will provide the necessary detailed drawings and specifications required to incorporate repair items c) (modified) and d) into the Rehabilitation Contract. Structural Research will provide assistance to the Structural Office in developing these details. Structural Research will also consider having the special brackets and jacking hardware fabricated beforehand to be loaned to the contractor during the work of removing the four steel hangar rods.

   R. Taylor also advised that he would arrange to be on-site during construction in 1987 to provide advice and assistance to the contractor during the most critical operations.

Minutes prepared by

O. E. Romakko

c.c. All present
R. Dorton
B. Bakht

Table 1/ Allowable Stresses for Select Structural Douglas Fir

AASHTO[1] Edition	Material Grade Select Structural (S.S.)	Extreme Fibre Bending (psi)[2]	Parallel to Grain		Horizontal Shear (psi)	Modulus of Elasticity (psi)x10^3
			Compression (psi)	Tension (psi)		
6th Edition 1953	Dense S.S.	2150	1550	2150[3]	145	1600
	S.S.	1900	1450	1900[3]	120	1600
8th Edition 1961	Dense S.S.	2150	1550	2150[3]	145	1600
	S.S.	1900	1450	1900[3]	120	1600
10th Edition 1969	Dense S.S.	2050	1500	2050[3]	120	1760
	S.S.	1900	1400	1900[3]	120	1760
11th Edition 1973	Dense S.S.	1900	1300	1100	85	1700
	S.S.	1600	1100	950	85	1600
12th Edition 1977	Dense S.S.	1900	1300	1100	85	1700
	S.S.	1600	1100	950	85	1600
13th Edition 1983	Dense S.S.	1900	1300	1100	85	1700
	S.S.	1600	1100	950	85	1600

1 See Reference 11.
2 Values tabulated as published in Imperial units.
3 Bending values specified for tension in earlier editions.

Table 2/ Specified Strengths for Select Structural Douglas Fir (MPa)

Reference Specification	Material Grade	Bending	Parallel to Grain		Horizontal Shear	Modulus of Elasticity
			Compression	Tension		
OHBDC [4]	Select Structural (S.S.)	24.0	16.5	13.0	1.1	11 000 (7 500)[1]
CSA-086 [12]	Select Structural Post & Beam	22.9	15.4	15.3	1.9	11 900
CSA-086 [12]	Select Structural Beam & Stringer	24.4	14.6	14.3	1.4	11 900

1 5th percentile modulus of elasticity

Table 3/ Assumed Specified Strengths for Select Structural Douglas Fir (MPa)

Reference Specification	Item	Bending	Parallel to Grain		Horizontal Shear	Modulus of Elasticity
			Compression	Tension		
—	Assumed Adjustment to Equivalent "First Growth" & "Dense" Material	+26.5%	+36.4%	+26.5%	+41.1	0.0%
OHBDC Adjusted	Assumed Specified Strengths	30.4	22.5	16.4	1.6	11 000 (7 500)
CSA-086[1] Adjusted	Assumed Specified Strengths, Post & Beam	29.0	21.0	19.4	2.7	11 900
CSA-086[1] Adjusted	Assumed Specified Strengths, Beam & Stringer	30.9	19.9	18.1	2.0	11 900

[1] Values from the CSA-086 used only in Truss Evaluation.

Table 4/ Factored Loads & Resistances, Wood Stringers

Span	Description	Force Effect	Total Factored Force Effect	Total Factored Member Resistance	Comments
South Approach Span 8.99 m	140 mm x 440 mm @ 457 mm O.C.	Moment (kN-M)	131.0	95.7 121.2	OHBDC Adjusted specified strength
		Shear (kN)	93.9	85.2 119.9 142.0 199.8	OHBDC Adjusted specified strength OHBDC - Assumed no end splits Adju. spec. str. & no end splits
North Approach Spans 4.57 m	140 mm x 340 mm @ 457 mm O.C.	Moment (kN-M)	60.4	56.7 71.8	OHBDC Adjusted specified strength
		Shear (kN)	73.8	74.6 105.0 124.3 175.0	OHBDC Adjusted specified strength OHBDC - Assumed no end splits Adj. spec. str. & no end splits
Main Span 2.29 m	114 mm x 285 mm @ 457 mm O.C.	Moment (kN-M)	29.9	36.9 46.7	OHBDC Adjusted specified strength
		Shear (kN)	57.9	56.0 78.7 93.2 131.1	OHBDC Adjusted specified strength Assumed no end splits Adj. spec. str. & no end splits

Table 5/ Distribution Widths (OHBDC), Wood Stringers

Span	Description	Distribution Width Dd (m)	
		Moment	Shear
South Approach Span	140 mm x 440 mm @ 457 mm O.C.	1.91	1.18
North Approach Spans	140 mm x 340 mm @ 457 mm O.C.	1.62	1.18
Main Span	114 mm x 285 mm @ 457 mm O.C.	1.50	1.18

Table 6/ Factored Loads & Resistances, Kingpost Floor Beams

	Maximum Total Factored Load Effect		Factored Member Resistance	
Design Assumptions	Two-Span Continuous Stringers	Simply Supported Stringers	OHBDC	Adjusted Specified Strengths (Table 3)
Moment in Wood Beam (kN-m)	157	140	164	208
Axial Compression in Wood Beam (kN)	688	612	2235	3048
Axial Compression in Wood Post (kN)	510	453	737	1005
Axial Tension, Net Section Steel Bars (kN)	897	797	466[1]	557[2]

[1] Steel Grade 230 MPa
[2] Steel Grade 275 MPa

Table 7/ Factored Load Effects, OHBDC Loading, Main Truss

Member[1] Number	Secondary Members Included						Secondary Members Excluded					
	Live Load		Dead Load		Total Load		Live Load		Dead Load		Total Load	
	Axial[2]	Moment[3]	Axial	Moment	Axial	Moment	Axial	Moment	Axial	Moment	Axial	Moment
2	-1362	55.7	-2089	83.9	-3451	139.6	-1385	—	-2119	—	-3504	—
13	-449	—	-503	—	-952	—	-699	—	-818	—	-1517	—
23	-283	—	-125	—	-408	—	-503	—	-304	—	-807	—
33	-352[4]	—	-69	—	-421	—	-424	—	-13	—	-437	—
32	-2049	25.9	-3109	32.8	-5158	58.7	-2027	35.2	-3081	34.2	-5108	69.4
34	+1963	97.2	+3049	54.6	+5012	151.8	+1989	106.5	+3073	44.2	+5062	150.7
5	+653	—	+926	—	+1579	—	+898	—	+1339	—	+2237	—
10	+278	—	+315	—	+593	—	+678	—	+894	—	+1572	—
15	+293	—	+322	—	+615	—	+662	—	+783	—	+1445	—
20	+165	—	+114	—	+279	—	+506	—	+470	—	+976	—
25	+200	—	+165	—	+365	—	+549	—	+437	—	+986	—
30	+116	—	+30	—	+146	—	+ 37	—	+199	—	+636	—
35	+146	—	+82	—	+228	—	+349	—	+179	—	+528	—

1 Member numbers identified in Figure 13.
2 Negative indicates compression. Units kN.
3 Units kN-m
4 Underlined values indicate OHBDC Truck Load governed.

Table 8/ Factored Loads & Resistances, Main Truss

Member	Factored Loading		Factored Resistances		Interaction	Conditions
	Axial (kN)	Moment (kN-m)	Axial (kN)	Moment (kN-m)		
2 End Diagonal	-3504	206	-6012	563	0.95	OHBDC
			-8197	713	0.72	OHBDC, Adjusted Specified Strengths
	-3504	135	-4315	640	1.02	CSA-086
			-5437	811	0.81	CSA-086, Adjusted Specified Strengths
32 Top Chord	-5108	265	-6012	563	1.32	OHBDC
			-8197	713	0.99	OHBDC, Adjusted Specified Strengths
	-5108	107	-4639	640	1.27	CSA-086
			-6323	811	0.94	CSA-086, Adjusted Specified Strengths
34 Bottom Chord	+5062	151	3158	686	1.82	OHBDC
			3984	869	1.44	OHBDC, Adjusted Specified Strengths
	+5062	151	4236	780	1.39	CSA-086
			5362	987	1.10	CSA-086, Adjusted Specified Strengths

Table 9/ Factored Loads & Resistances, Main Truss

Member	Factored Loading		Factored Resistances		Interaction	Conditions
	Axial (kN)	Moment (kN-m)	Axial (kN)	Moment (kN-m)		
13 Primary Compression Diagonal	-1517	187	-2954	221	1.36	OHBDC
			-4029	280	1.05	OHBDC, Adjusted Specified Strengths
	-1517	55	-1397	209	1.35	CSA-086
			-1397	265	1.29	CSA-086, Adjusted Specified Strengths
	-1517	49	-2954	221	0.74	Repeat, respectively, the four conditions above. Assuming secondary diagonals provide support at mid-length.
			-4029	280	0.55	
	-1517	40	-2404	209	0.82	
			-3279	265	0.62	
23 Primary Compression Diagonal	-807	magni-fication factor too large	-2076	133	—	OHBDC
			-2831	169	—	OHBDC, Adjusted Specified Strengths
	-807	26.5	-396	103	2.30	CSA-086, Adjusted
			-396	131	2.24	CSA-086 Specified Strengths
	-807	27.3	-2076	133	0.59	Repeat, respectively, the four conditions above. Assuming secondary diagonals provide support at mid-length.
			-2831	169	0.45	
	-807	18.2	-1569	103	0.69	
			-1582	131	0.65	

Table 10/ Factored Loads & Resistances, Main Truss

Member	Factored Loading		Factored Resistances		Interaction	Conditions
	Axial (kN)	Moment (kN-m)	Axial (kN)	Moment (kN-m)		
33 Primary Compression Diagonal	-437	4	-2076	133	0.69	OHBDC
			-2831	169	0.54	OHBDC, Adjusted Specified Strengths
	-437	15	-369	103	1.32	CSA-086
			-369	131	1.30	CSA-086, Adjusted Specified Strengths
	-437	12	-2076	133	0.30	Repeat, respectively, the four conditions above. Assuming secondary diagonals provide support at mid-length.
			-2831	169	0.23	
	-437	10	-1471	103	0.39	
			-1471	131	0.37	

Table 11/ Factored Loads & Resistances, Steel Bars, Main Truss

Member Number	Factored Loading		Factored Resistances	
	Secondaries Included (kN)	Secondaries Excluded (kN)	Fy = 230 MPa (kN)	Fy = 275 MPa (kN)
5	1579	2237	1851	2213
10	593	1572	1210	1447
15	615	1445	1210	1447
20	279	976	699	836
25	365	986	699	836
30	146	636	422	505
35	228	528	519	621

Table 12/ Shear Due to Steel Keyways in Bottom Chord (kN)

	Factored Load Effect	Factored Member Resistance		
		No Adjustments to Specifications	Adjusted Specified Strengths	Comments
Horizontal Shear in Bottom Chord	2254	872	1254	CSA-086
Horizontal Shear in Steel Keyways	2254	14 020	—	OHBDC
Bearing of Keyways on Bottom Chord	2254	2 489	3395	OHBDC

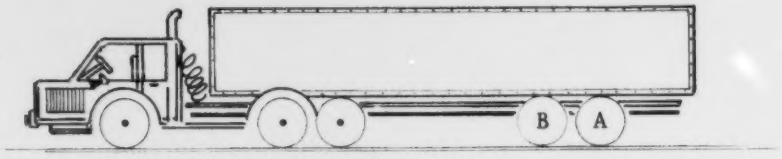
Table 13/ Summary Load/Strength Interactions[2], Main Truss

Member	OHBDC	CSA-086	OHBDC[1]	CSA-086[1]	
2	0.95	1.02	0.72	0.81	
13	1.36 0.74	1.35 0.82	1.05 0.55	1.29 0.62	without secondary support with secondary support
23	— 0.59	2.30 0.69	-- 0.45	2.24 0.65	without secondary support with secondary support
33	0.69 0.30	1.32 0.39	0.54 0.23	1.30 0.37	without secondary support with secondary support
32	1.32	1.27	0.99	0.94	
34	1.82	1.39	1.44	1.10	
Bottom Chord Horizontal Shear in Wood	--	2.58	--	1.80	
5	1.21	—	1.01	--	
10	1.30	--	1.09	--	
15	1.19	--	1.00	--	
20	1.40	—	1.17	--	
25	1.41	—	1.18	--	
30	1.51	—	1.26	--	
35	1.02	—	0.85	—	

1 Adjusted specified wood strengths and higher yield strength of steel where applicable.

2 For single force mode tabulated value is load divided by resistance.

Table 14/ Vehicle Weights, Preliminary Testing, Main Span Deck



Load Level	Steering Axle (kN)	Front Tandem Total (kN)	Rear Tandem Total (kN)
1	47.9	181	195
2	46.9	146	231
3	43.9	100	280

Table 15/ Steel Strains in Floor Beams, Preliminary Testing, Main Span Deck

Rear Tandem at Floor Beam Number	Axle Pos.	Strain in Steel Bars of Floor Beam # (με)							
		1	2	3	4	5	6	7	8
1	A	343	76	-22	-5	3	-	-	-
	AB	392	189	-10	-4	1	-	-	-
	B	444	298	25	2	-4	-	-	-
2	A	360	364	76	13	-6	-6	-	-
	AB	198	407	191	29	-17	-5	-	-
	B	59	351	380	65	-28	-7	-	-
3	A	12	299	456	91	-30	-8	-3	-
	AB	-11	220	457	181	-10	-5	-6	-
	B	53	108	430	299	49	6	-7	-
4	A	89	98	360	338	87	14	-7	-1
	AB	166	102	178	382	227	45	-16	0
	B	182	121	69	348	390	77	-27	-3
5	A	156	134	34	302	472	104	-29	-5
	AB	80	158	45	213	468	232	-8	-2
	B	32	135	83	126	452	335	29	2
(Vehicle in Lane 2)									

Table 16/ Deck Grid Analogy, GRID1

Rear Tandem at Floor Beam Number	Axle Pos.	Strain in Steel Bars of Floor Beam # (με)							
		1	2	3	4	5	6	7	8
2	B	46	311	262	53	-22	-5	2	1
3	A	9	233	367	72	-30	-6	3	1
	AB	-11	167	346	165	-18	-4	2	0
	B	29	83	362	232	9	2	-1	0
4	A	69	71	255	308	43	9	-3	-1
	AB	128	77	127	333	137	27	-12	-2
	B	136	96	41	310	263	53	-22	-4
5	A	108	107	17	234	367	71	-30	-5
	AB	55	120	23	174	343	165	-18	-3
	B	18	108	59	90	360	231	9	1

Table 17/ Deck Grid Analogy, GRID2

Rear Tandem at Floor Beam Number	Axle Pos.	Strain in Steel Bars of Floor Beam # (με)							
		1	2	3	4	5	6	7	8
2	B	46	315	269	54	-24	-5	2	1
3	A	8	236	376	73	-33	-7	3	1
	AB	-13	170	355	168	-21	-4	2	1
	B	27	85	371	235	8	2	-1	0
4	A	68	73	261	312	43	9	-4	-1
	AB	130	79	130	336	141	29	-13	-2
	B	139	97	41	314	269	54	-24	-4
5	A	111	108	16	237	375	73	-33	-7
	AB	57	121	22	177	352	168	-20	-4
	B	18	109	59	92	368	234	8	2

Table 18/ Grid vs 3-D Frame

	Rear Tandem at Floor Beam Number	Axle Pos.	Strain in Steel Bars of Floor Beam # (με)							
			1	2	3	4	5	6	7	8
Grid Analogy GRID2	2	B	46	315	269	54	-24	-5	2	1
	3	A	8	236	376	73	-33	-7	3	1
		AB	-13	170	355	168	-21	-4	2	1
		B	27	85	371	235	8	2	-1	0
3-Dimensional Frame Analysis	2	B	53	293	273	74	-29	-7	3	0
	3	A	17	243	375	94	-37	-9	4	0
		AB	-7	183	361	181	-15	-3	2	0
		B	16	107	370	242	19	5	-2	0

Table 19/ Deck Grid Analogy, GRID3

Rear Tandem at Floor Beam Number	Axle Pos.	Strain in Steel Bars of Floor Beam # (με)							
		1	2	3	4	5	6	7	8
2	B	47	348	290	56	-25	-5	2	0
3	A	8	258	409	76	-34	-7	3	1
	AB	-14	184	386	183	-23	-4	2	0
	B	31	88	404	257	8	2	-1	0
4	A	76	76	282	345	44	9	-4	-1
	AB	142	83	139	373	150	29	-13	-3
	B	152	105	42	347	291	56	-25	-5
5	A	120	118	15	259	409	76	-34	-7
	AB	60	135	22	192	383	182	-22	-4
	B	18	121	65	95	401	256	8	2

Table 20/ Deck Grid Analogy, GRID4

Rear Tandem at Floor Beam Number	Axle Pos.	Strain in Steel Bars of Floor Beam # (με)							
		1	2	3	4	5	6	7	8
2	B	1	440	272	1	0	0	0	0
3	A	1	272	440	1	0	0	0	0
	AB	11	133	437	133	0	0	0	0
	B	75	1	440	272	1	0	0	0
4	A	113	1	272	440	1	0	0	0
	AB	168	31	133	437	133	0	0	0
	B	157	93	1	440	272	1	0	0
5	A	112	134	1	272	440	1	0	0
	AB	40	176	31	133	437	133	0	0
	B	0	157	93	1	440	272	1	0

Table 21/ Deck Grid Analogy, Grid Combination

Rear Tandem at Floor Beam Number	Axle Pos.	Strain in Steel Bars of Floor Beam # (με)							
		1	2	3	4	5	6	7	8
2	B	47	348	290	56	-25	-5	2	0
3	A	1	272	440	1	0	0	0	0
	AB	11	133	437	133	0	0	0	0
	B	75	1	440	272	1	0	0	0
4	A	76	76	282	345	44	9	-4	-1
	AB	142	83	139	373	150	29	-13	-3
	B	152	105	42	347	291	56	-25	-5
5	A	112	134	1	272	440	1	0	0
	AB	40	176	31	133	437	133	0	0
	B	0	157	93	1	440	272	1	0

Table 22/ Deck Grid Analogy, GRIDN1

Rear Tandem at Floor Beam Number	Axle Pos.	Strain in Steel Bars of Floor Beam # ($\mu\varepsilon$)							
		1	2	3	4	5	6	7	8
2	B	121	296	245	79	1	-10	-4	0
3	A	88	249	290	113	8	-11	-6	-1
	AB	51	187	297	180	36	-8	-8	-2
	B	50	134	291	242	78	1	-10	-4
4	A	54	109	248	287	112	8	-11	-6
	AB	76	96	194	294	178	35	-9	-8
	B	82	99	142	289	239	77	0	-10
5	A	74	100	116	245	285	112	8	-11
	AB	57	102	98	191	293	178	35	-8
	B	37	95	97	140	288	240	78	1

Table 23/ Vehicle Weights, Proof Testing, Deck System

Load Level	Vehicle #1			Vehicle #2		
	Steering Axle (kN)	Front Tandem (kN)	Rear Tandem (kN)	Steering Axle (kN)	Front Tandem (kN)	Rear Tandem (kN)
1	48.1	181	194	only Vehicle #1 used for		
2	46.3	146	231	first three load levels		
3	43.9	100	280	--	--	--
4	49.6	209	314	51.8	198	313
5	51.1	240	356	54.4	227	356
6	51.9	256	377	55.8	241	378
7	52.6	268	402	56.9	253	402
8	52.5	266	423	56.6	250	424
9	52.0	256	451	56.6	249	443

Table 24/ Steel Strains in Floor Beams, TEST1
Main Span Deck

Rear Tandem at Floor Beam Number	Axle Pos.	Strain in Steel Bars of Floor Beam # (με)							
		1	2	3	4	5	6	7	8
1	A	125	57	10	0	0	0	0	0
	AB	172	124	36	1	-7	0	0	0
	B	186	176	61	7	-9	-4	0	0
2	A	173	216	98	17	-6	0	0	0
	AB	117	243	168	34	-6	0	0	0
	B	63	227	241	62	2	-7	0	0
3	A	52	214	264	77	9	0	0	0
	AB	29	163	294	125	34	0	0	0
	B	35	117	275	183	75	10	-8	0
4	A	52	100	240	214	114	26	-6	0
	AB	68	88	179	232	177	46	-7	0
	B	75	93	125	221	248	76	-3	0
5	A	72	96	107	206	277	102	0	0
	AB	48	97	90	158	300	165	15	4
	B	33	88	99	115	270	230	42	12

Table 25/ Steel Strains in Floor Beams, TEST2
Main Span Deck

Rear Tandem at Floor Beam Number	Strain in Steel Bars of Floor Beam # (με)							
	1	2	3	4	5	6	7	8
1	472	508	328	340	351	183	81	18
2	210	577	567	319	396	327	165	82
3	27	262	634	515	340	392	334	329
4	-4	47	227	529	622	384	363	305
5	-1	-16	12	258	665	574	315	362

Table 26/ Factored Loads & Resistances Kingpost Floorbeams

	Total Factored Load Effect	Factored Member Resistances	
		OHBDC	Adjusted
Moment in Wood Beam (kN - m)	91	164	208*
Axial Compression in Wood Beam (kN)	524	2235	3048*
Axial Tension in Steel Bars (kN)	550	466	432**

* Adjusted Specified Strengths, see Section 5.

** Reduced cross section due to assumed deterioration.

Table 27/ Distribution Widths (OHBDC), South Approach Span Stringers

	Distribution Width (m)	
	Moment	Shear
Span with New Deck	1.52	1.18
Span with Old Deck	1.91	1.18

Table 28/ Factored Loads & Resistances, South Approach Span Stringers

	Total Factored Load Effect	Factored Load Effect/Stringer[1]	Factored Member Resistance/Stringer	Comments
Moment (kN-m)	171	143	86	OHBDC
			109	Adjusted Specified Strength
Shear (kN)	98	98	85	OHBDC
			120	Adjusted Specified Strength
			142	Assumed No End Splits

[1] Amount assumed to be carried by stringer according to relative stiffness between stringer and deck

Table 29/ Factored Loads and Resistances, South Approach Span Stringers

	Total Factored Load Effect Per Member	Factored Load Effect Per Stringer	Total Factored Resistance Per Member	Factored Resistance Per Stringer	Comments
GRIDS2 MODEL	157	116	179	86	OHBDC, no modification
			204	109	Adjusted Specified Strength

Table 30/ Vehicle Weights, Main Span Truss Testing

Load Level	Vehicle #1			Vehicle #2		
	Steering Axle kN	Front Tandem kN	Rear Tandem kN	Steering Axle kN	Front Tandem kN	Rear Tandem kN
1	47.9	177	180	50.1	181	182
2	50.4	225	223	54.4	226	226
3	52.7	270	268	58.6	271	270
4	54.9	314	316	62.9	317	314
5	57.2	359	362	66.9	359	361
6	58.6	386	389	69.5	387	386
7	60.1	416	414	71.2	405	405

Table 31/ Projected Member Strains, Main Span Truss

Member Number (see Figure 13)	Average Live Load Strain[1] με	Assumed Dead Load Strain με	Total Member Strains με	Assumed Failure Strain[2] με
Wood Members				
2	-69	-331	-400	
13	-72	-180	-252	
23	-132	-95	-227	1500
33	-84	-5	-89	
32	-165	-481	-646	
34	162	434	596	1150
Steel Members				
5	104	461	565	894
10	139	460	599	874
15	156	403	559	
20	258	400	658	834
25	339	371	710	
30	361	274	615	816
35	186	207	393	842

[1] Load Level 4, two vehicle loading, longitudinal position 3

[2] Steel Values adjusted to reflect effects of the net sections.

Table 32/ Vertical Steel Bar Stresses During Testing

Member Number (see Figure 13)	Unfactored Dead Load Stress[1] MPa	Stress Range[2] Available for Live Load MPa	Maximum Stress[3] Monitored for Load Level 4 MPa	Project Stress for Load Level 5 MPa
5	92	87	37	42
10	92	83	50	57
15	81	94	54	61
20	80	87	83	94
25	74	93	89	101
30	55	108	93	106
35	41	127	49	56

[1] From from analysis Section 5, no secondary members
[2] Difference between dead load stress and assumed yield of 230 MPa adjusted for net section.
[3] Converted from measured strains. Peak value is given, not average of three bars.

Table 33/ Considerations for Reductions in Factored Load Effects

Member Number	% Overload[1] Preliminary Evaluation	% Reduction in Total Factored Load Effects		
		One OHBDC Truck Only	No Factor on Dead Load of Wood Components	Combined effect One OHBDC Truck & Reduced Dead Load
Bottom Chord End Shear	44.%	5.3%	7.6%	12.7%
34	10% approx.	5.3%	8.4%	12.7%
5	17.3%	5.3%	7.6%	12.8%
10	23.1%	5.9%	7.1%	13.0%
15	16.3%	6.4%	6.8%	13.2%
20	28.4%	7.4%	5.8%	13.2%
25	29.0%	8.2%	5.5%	13.7%
30	33.6%	10.4%	4.0%	14.5%
35	1.2%	14.5%	4.4%	18.9%

[1] With respect to the factored load effect

Table 34/ Recommended Bar Replacement Sizes and Jacking Capacities

Member Number (see Fig. 13)	Recommended Thread Bar (400 MPa)	Theoretical[1] Member Dead Load (kN)	Minimum Jack Capacity Per Bar (kN)	Maximum[2] Jacking Force Per Bar (kN)	Total Bar[2] Elongation Associated with Maximum Jacking Force (mm)
5	3 @ 55M	1060	530	620	5
10	3 @ 45M	710	360	400	6
15	3 @ 45M	620	310	400	6
20	3 @ 35M	370	190	240	7
25	3 @ 35M	350	175	240	7
30	3 @ 30M	160	80	140	7
35	3 @ 30M	140	70	170	7

[1]High value estimates. Total force per three bars.
[2]These values reflect the properties of the existing bars.

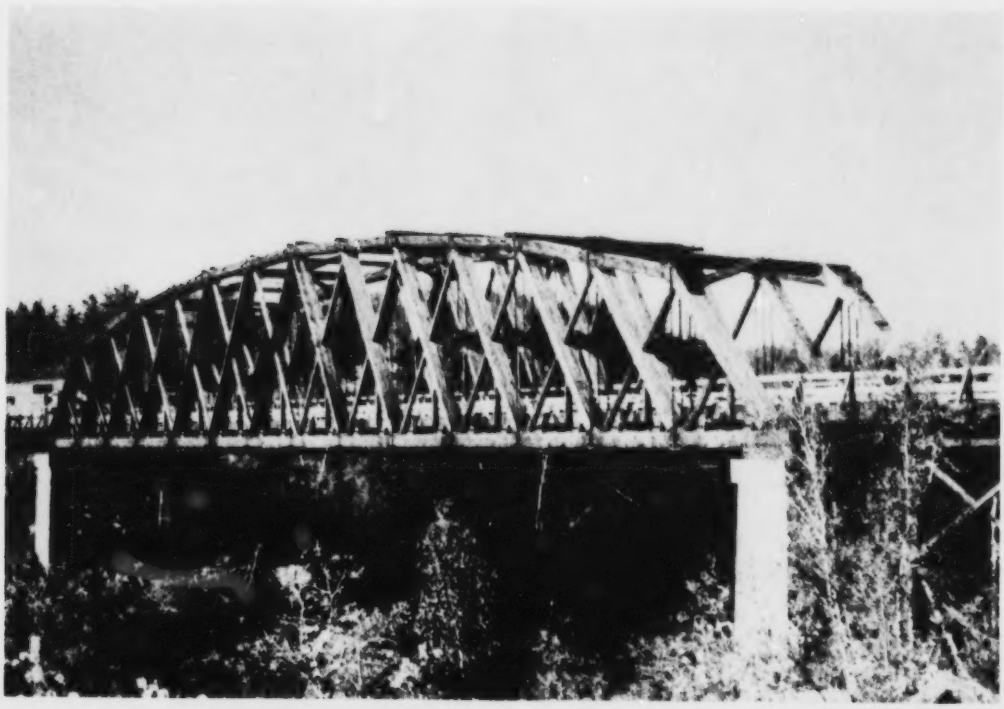


Figure 1/ Sioux Narrows Bridge, Hwy. 71, Kenora Ontario

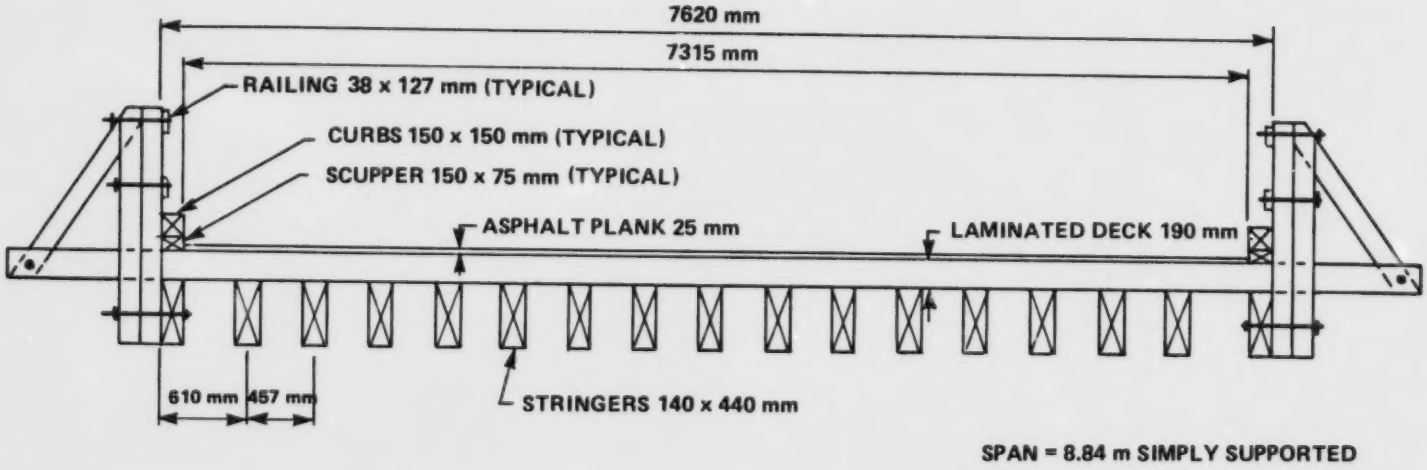

7620 mm
7315 mm
RAILING 38 x 127 mm (TYPICAL)
CURBS 150 x 150 mm (TYPICAL)
SCUPPER 150 x 75 mm (TYPICAL)
ASPHALT PLANK 25 mm
LAMINATED DECK 190 mm
610 mm
457 mm
STRINGERS 140 x 440 mm
SPAN = 8.84 m SIMPLY SUPPORTED

Figure 2/ Cross-section, South Approach Span

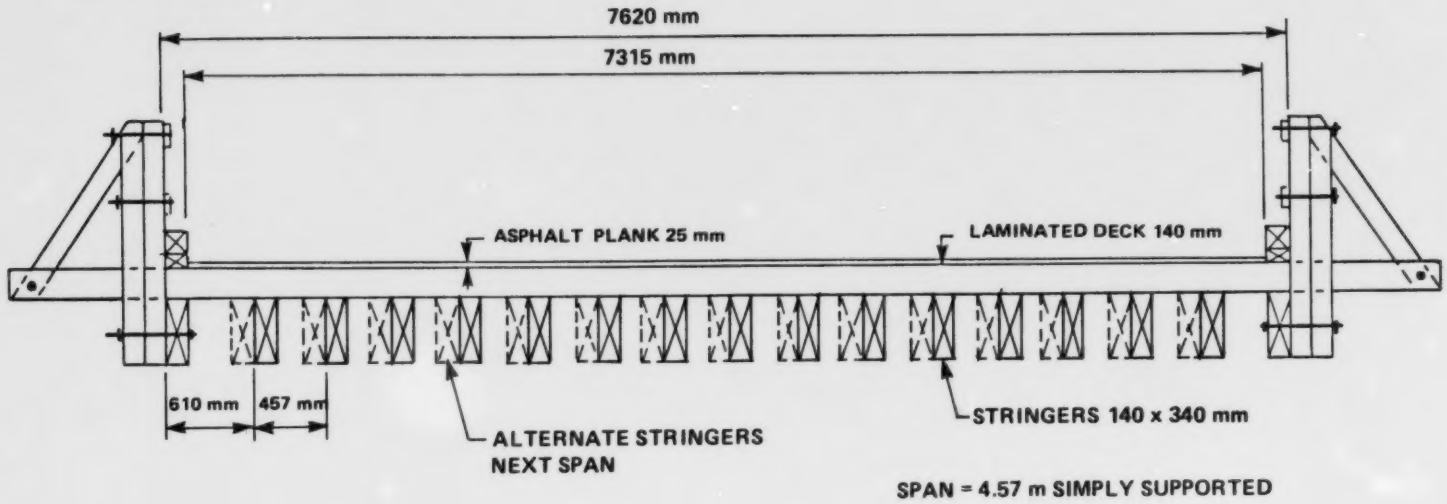


Figure 3/ Cross-section, North Approach Spans

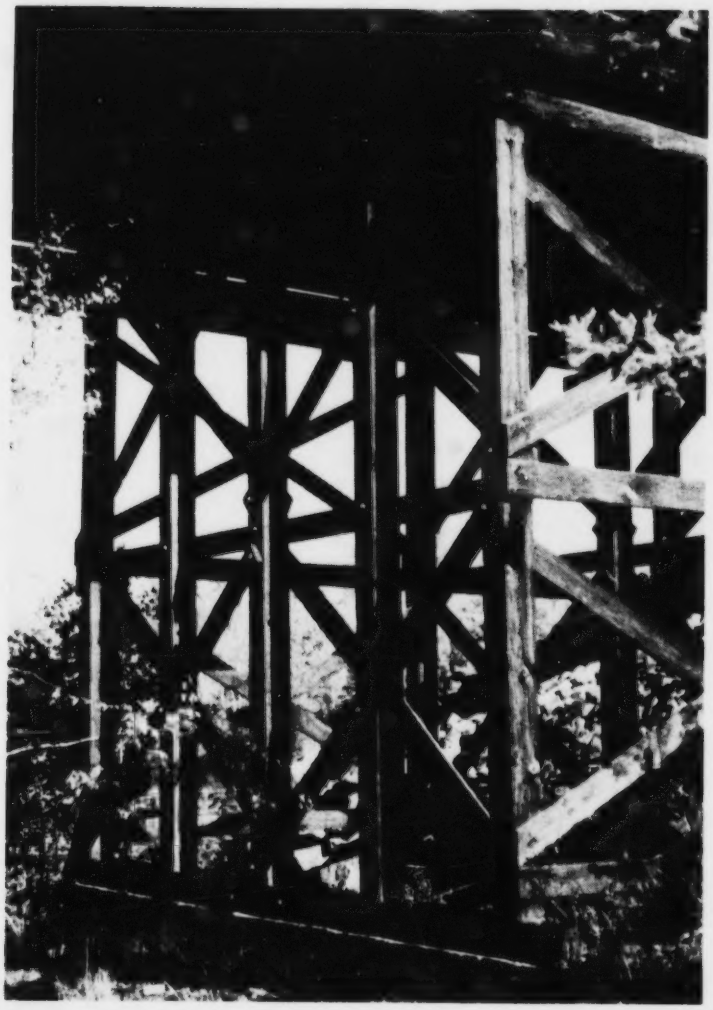
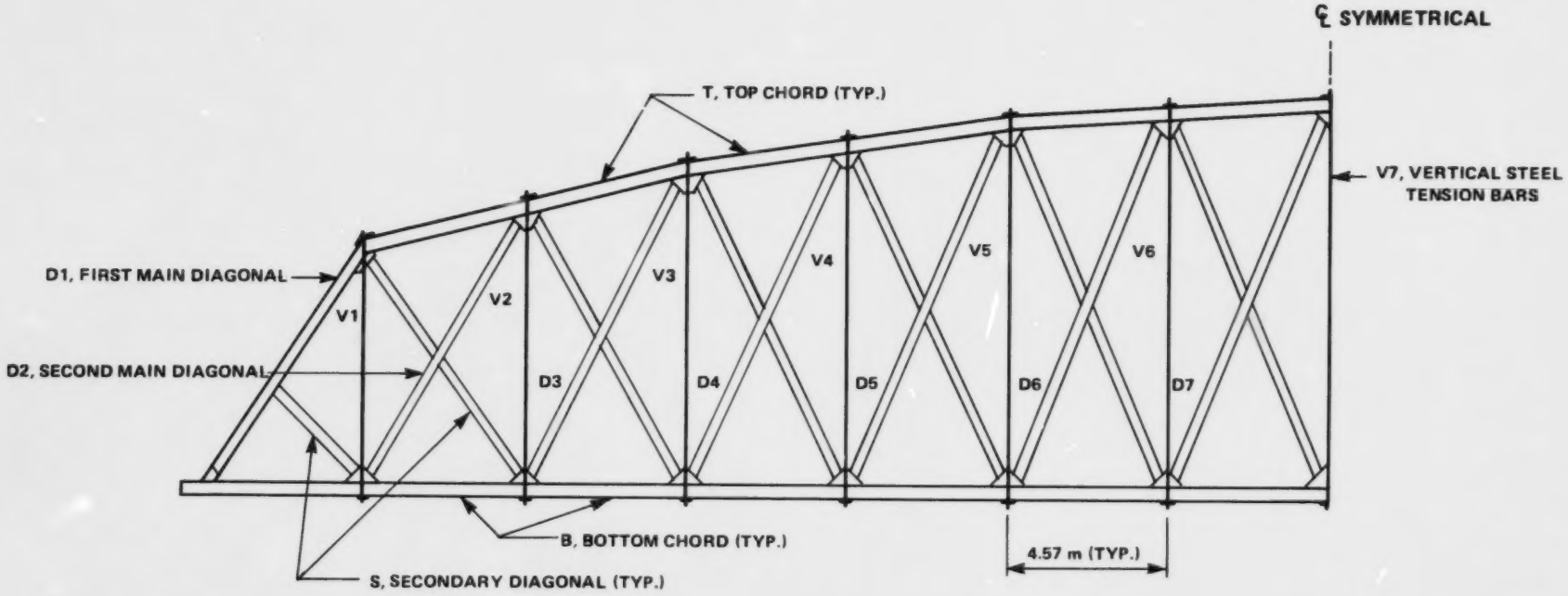


Figure 4/ North Approach Spans, Trestle Supports



MEMBER	SIZE	MEMBER	SIZE
T&D1	4 @ 190 x 483 mm depth	S	184 x 235 mm depth
B	4 @ 190 x 533 mm depth	V1	3 @ 70 mm ϕ Steel Bars
D2	2 @ 289 mm & 1 @ 184 mm, x 394 mm depth	V2 & V3	3 @ 57 mm ϕ Steel Bars
D3	2 @ 289 mm & 1 @ 184 mm, x 343 mm depth	V4 & V5	3 @ 44 mm ϕ Steel Bars
D4	2 @ 289 mm & 1 @ 184 mm, x 279 mm depth	V6	3 @ 35 mm ϕ Steel Bars
D5, D6 & D7	2 @ 289 mm & 1 @ 184 mm, x 241 mm depth	V7	3 @ 38 mm ϕ Steel Bars

Figure 5/ Truss Member Sizes, Main Span

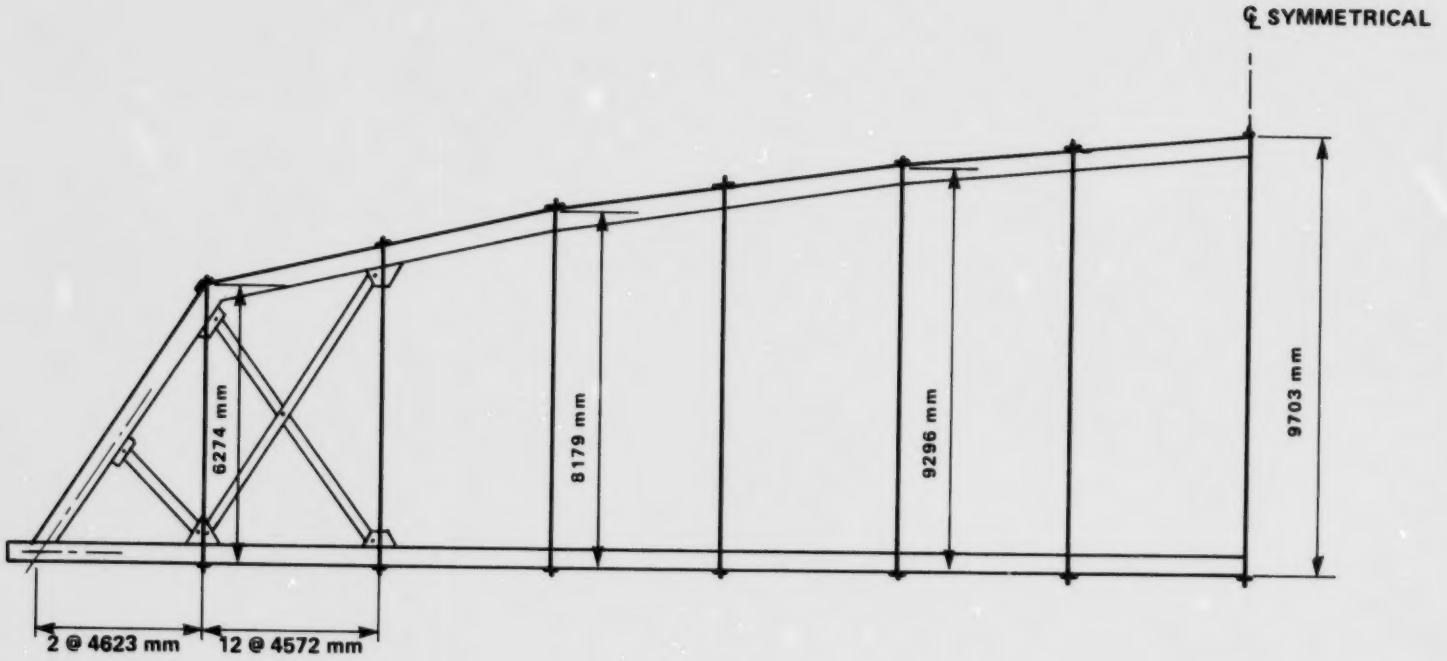


Figure 6/ Overall Dimensions, Main Span Truss

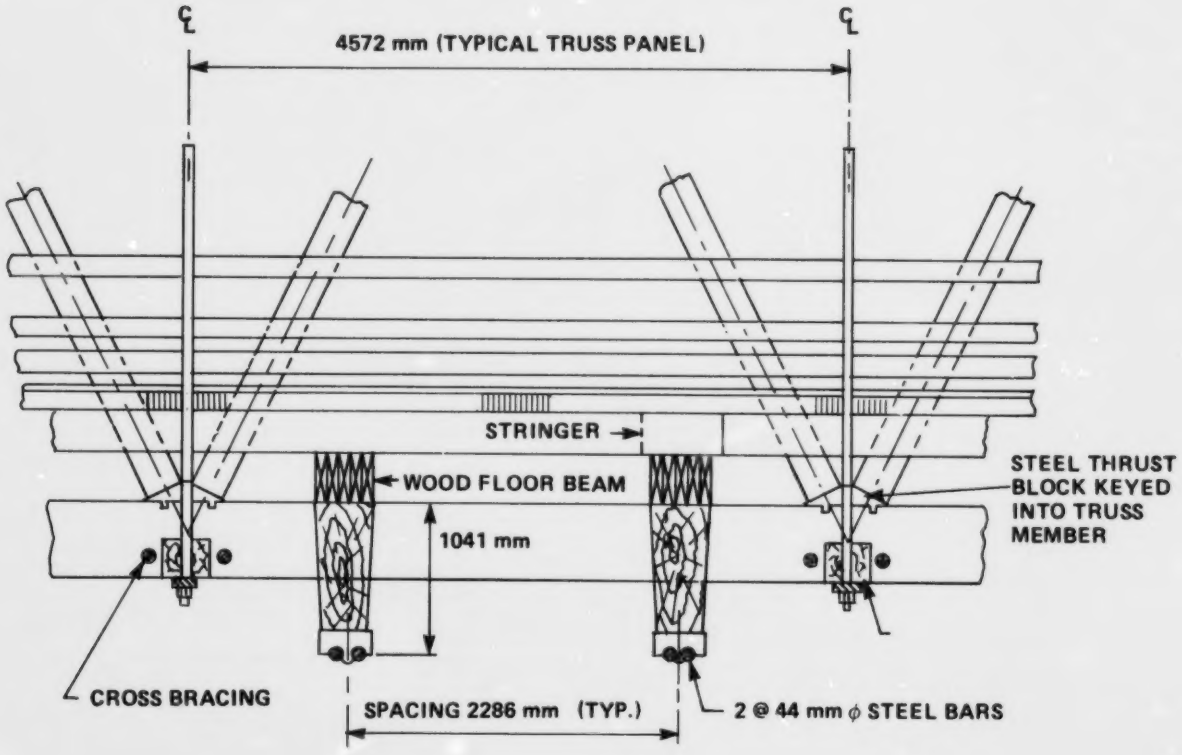


Figure 7/ Partial Elevation, Section A-A (Figure 8)

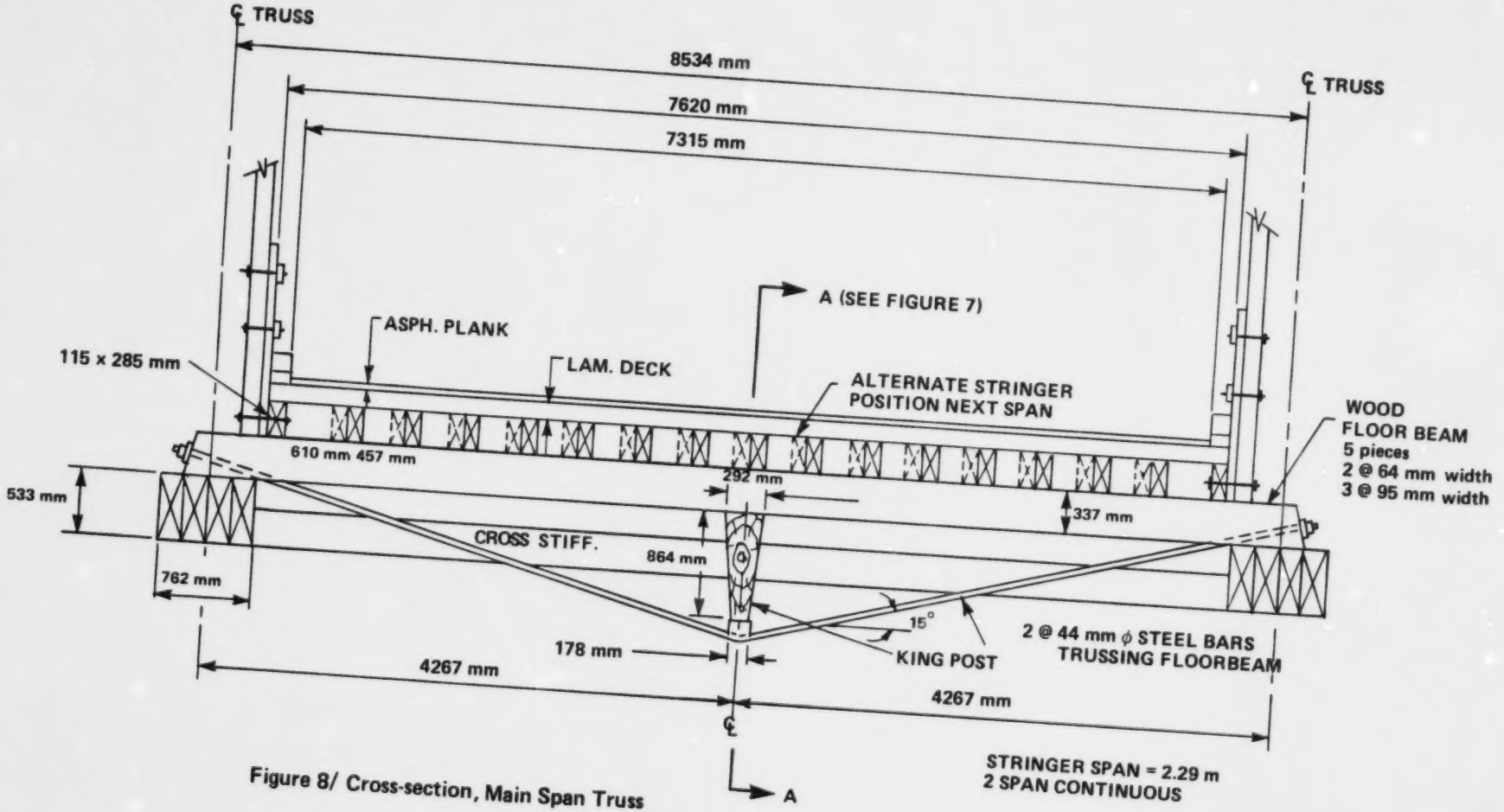


Figure 8/ Cross-section, Main Span Truss

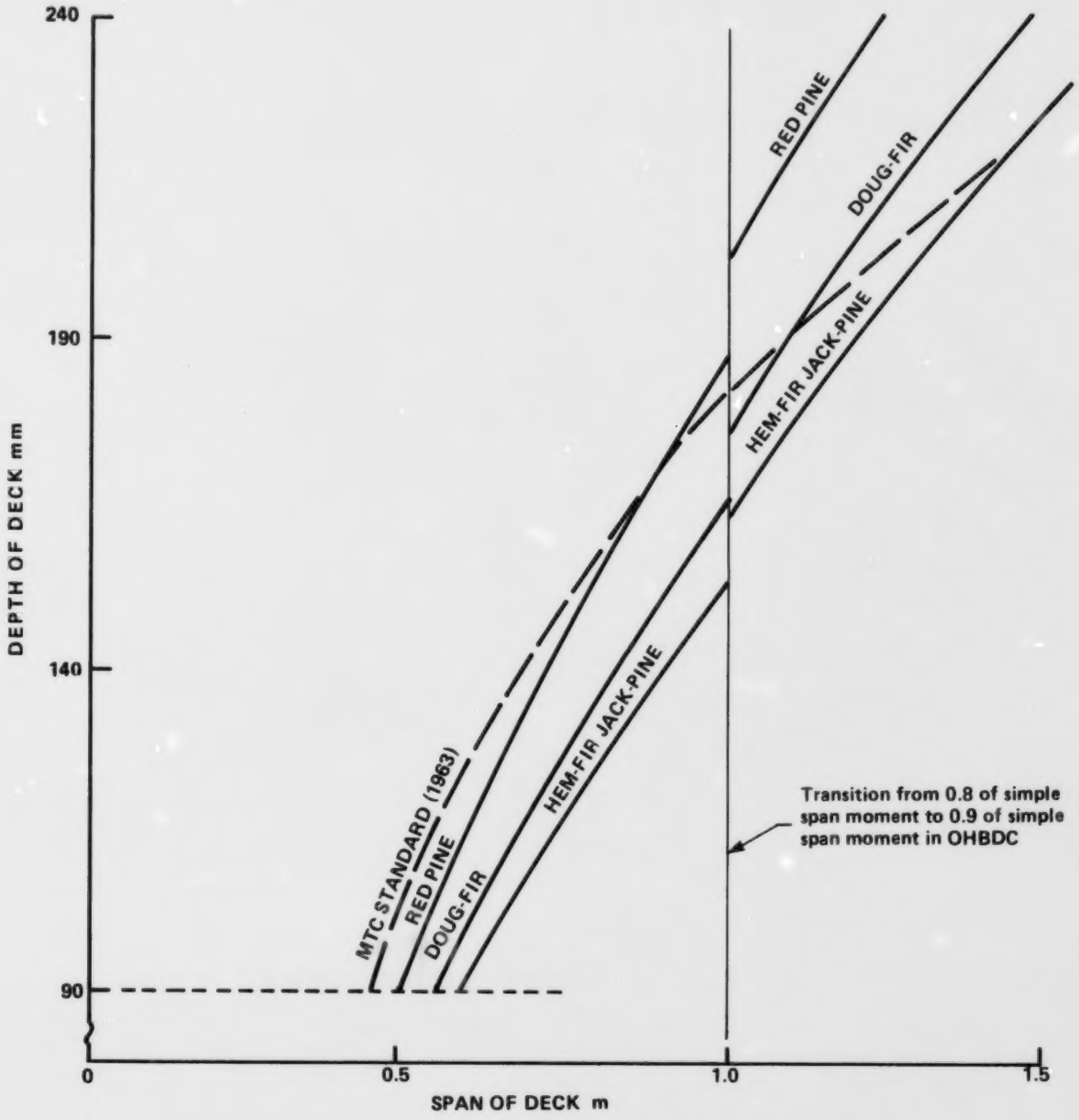


Figure 9/ Span vs Depth, Transverse Laminated Decks by the OHBDC (Reference 14)

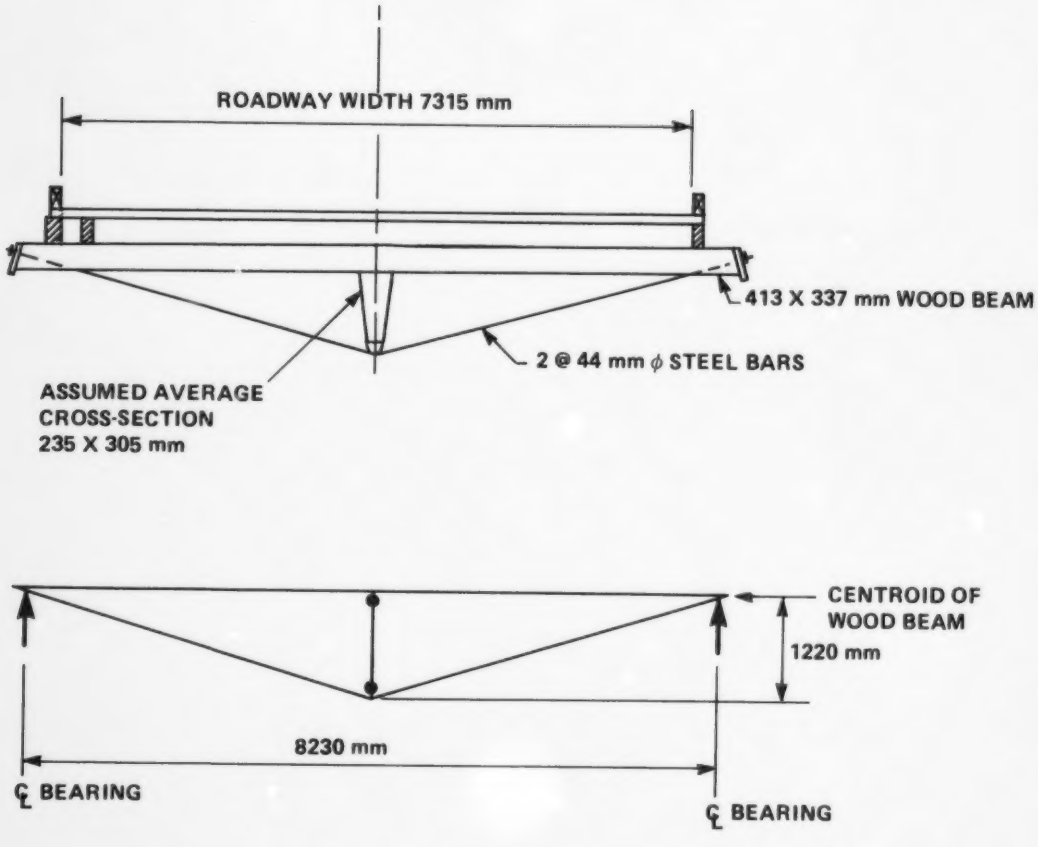
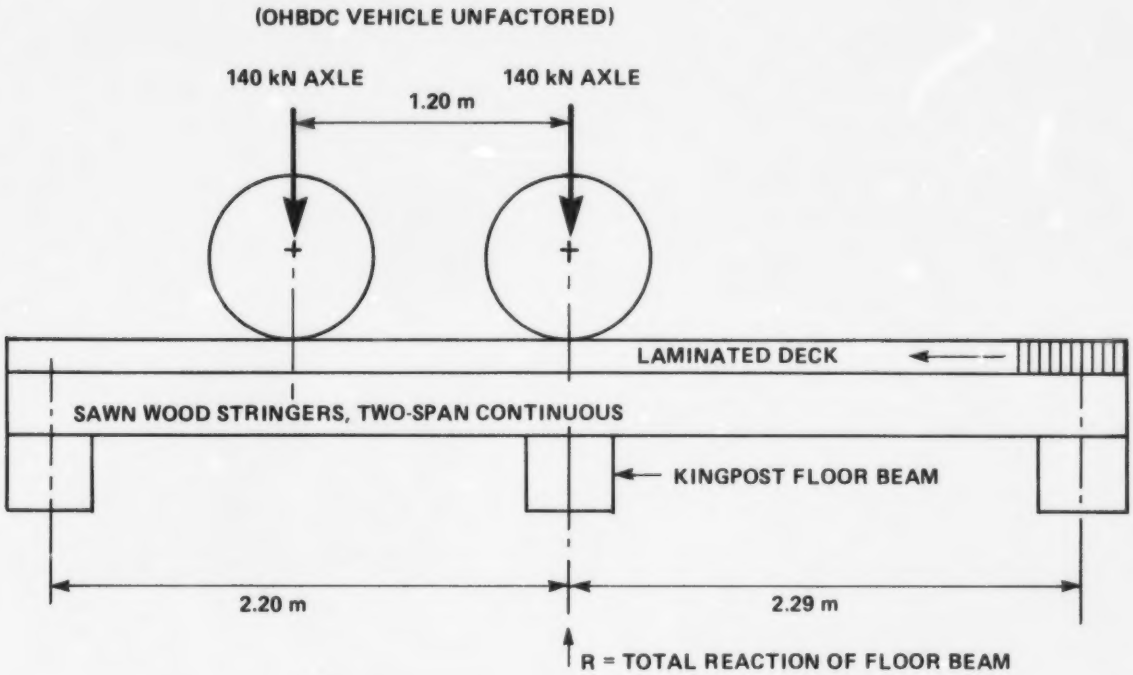


Figure 10/ Frame Simulation, Kingpost Floor Beam



ASSUMING CONTINUITY OF STRINGER $R = 140\ kN + (.659)^* 140\ kN$

$R = 232.3\ kN/VEHICLE$

ASSUMING SIMPLY SUPPORTED SPANS $R = 140 + \left(\frac{1.09}{2.29}\right) 140\ kN$

$R = 206.6\ kN/VEHICLE$

(* – INFLUENCE TABLES)

Figure 11/ Longitudinal Axle Distribution, Kingpost Floor Beam

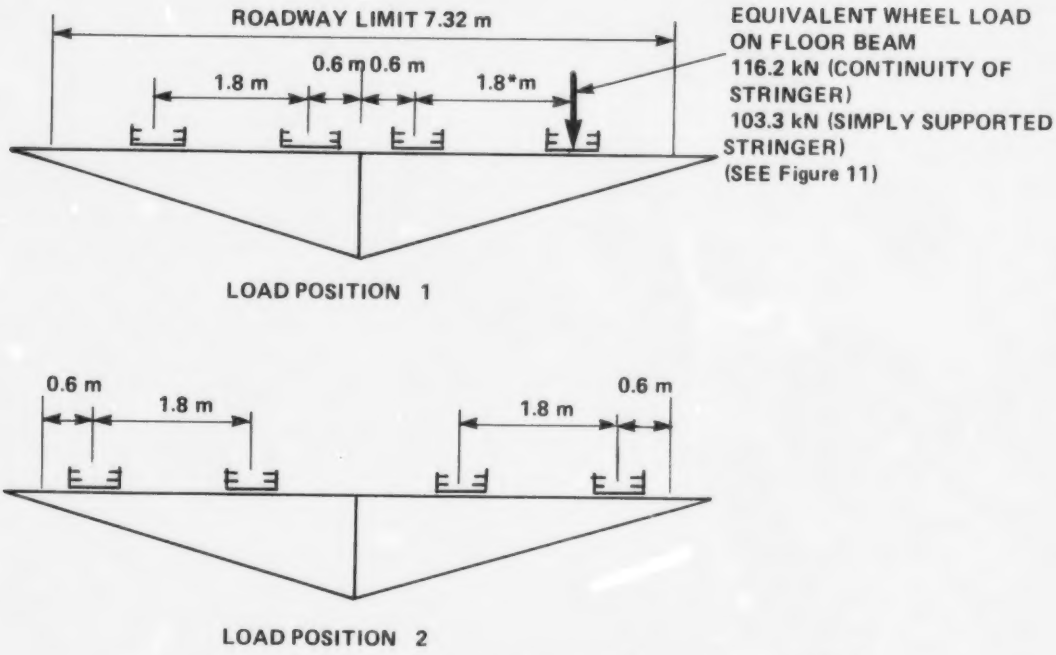


Figure 12/ Transverse Vehicle Positions, Kingpost Floor Beam

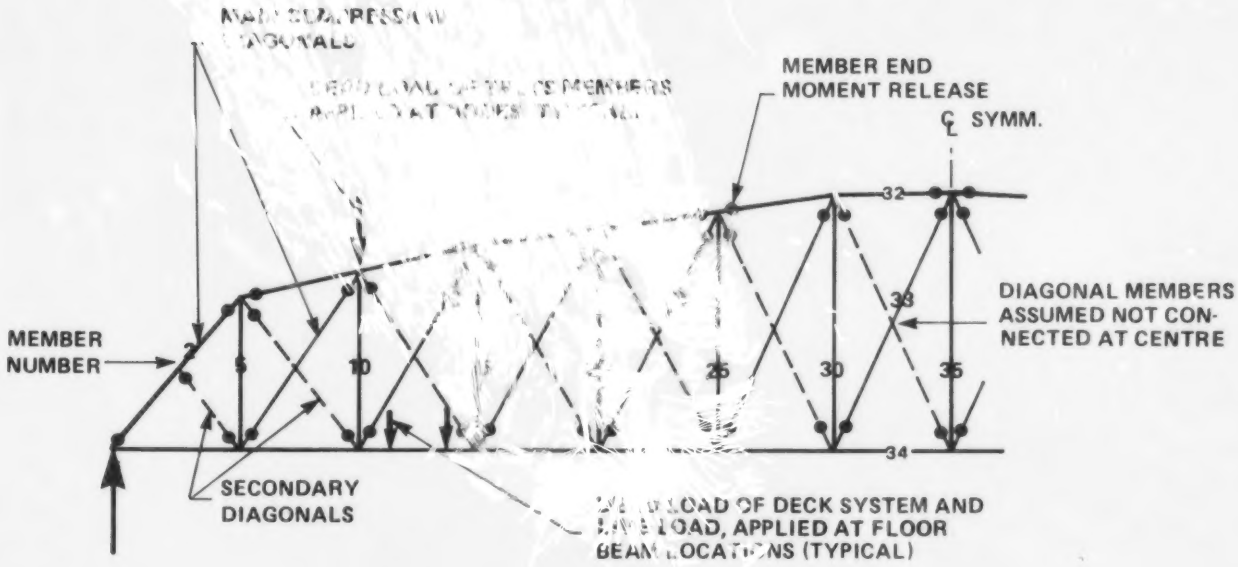
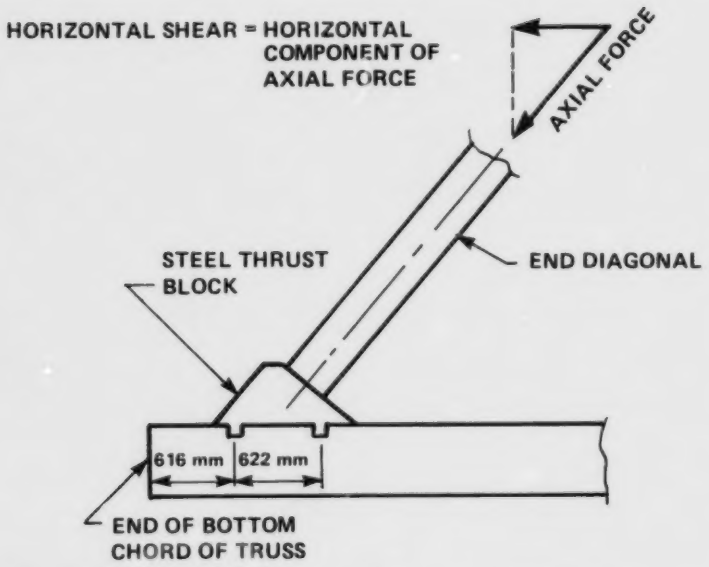


Figure 13/ Frame Analogy, Main Span Truss



LENGTH OF SHEAR PLANE IN WOOD = 616 + 622 = 1238 mm

Figure 14/ Steel Thrust Block, Main Span Truss

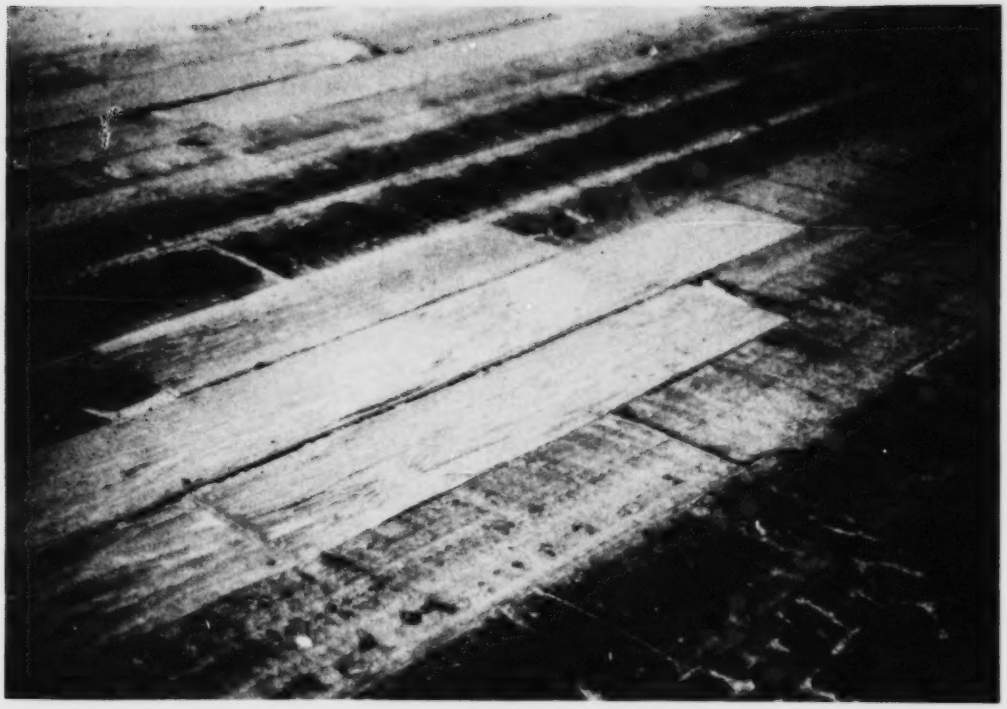


Figure 15/ Old Deteriorated Wearing Surface

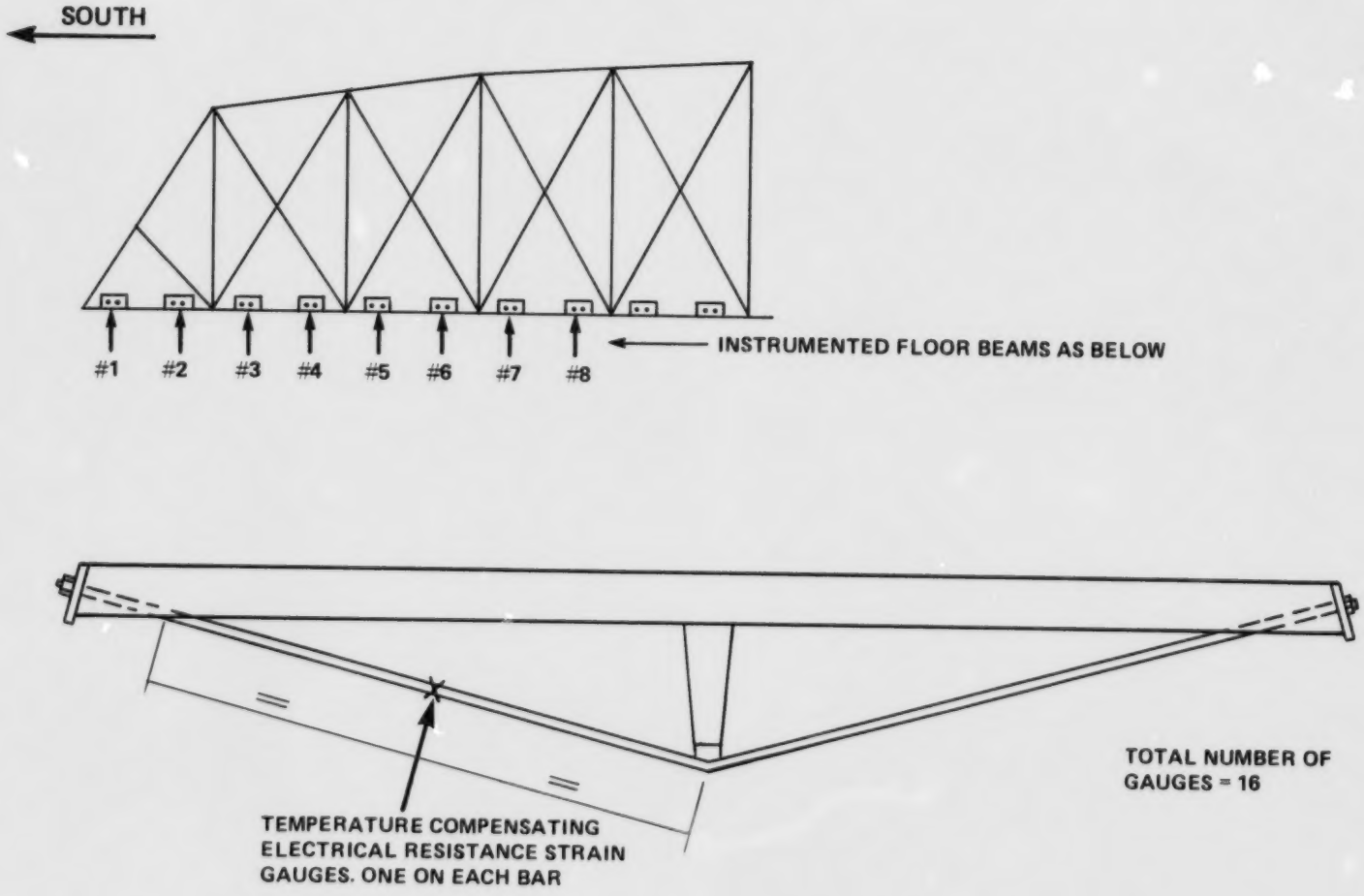


Figure 16/ Instrumentation, Kingpost Floor Beams

Figure 17/ MTC Load Testing Vehicle

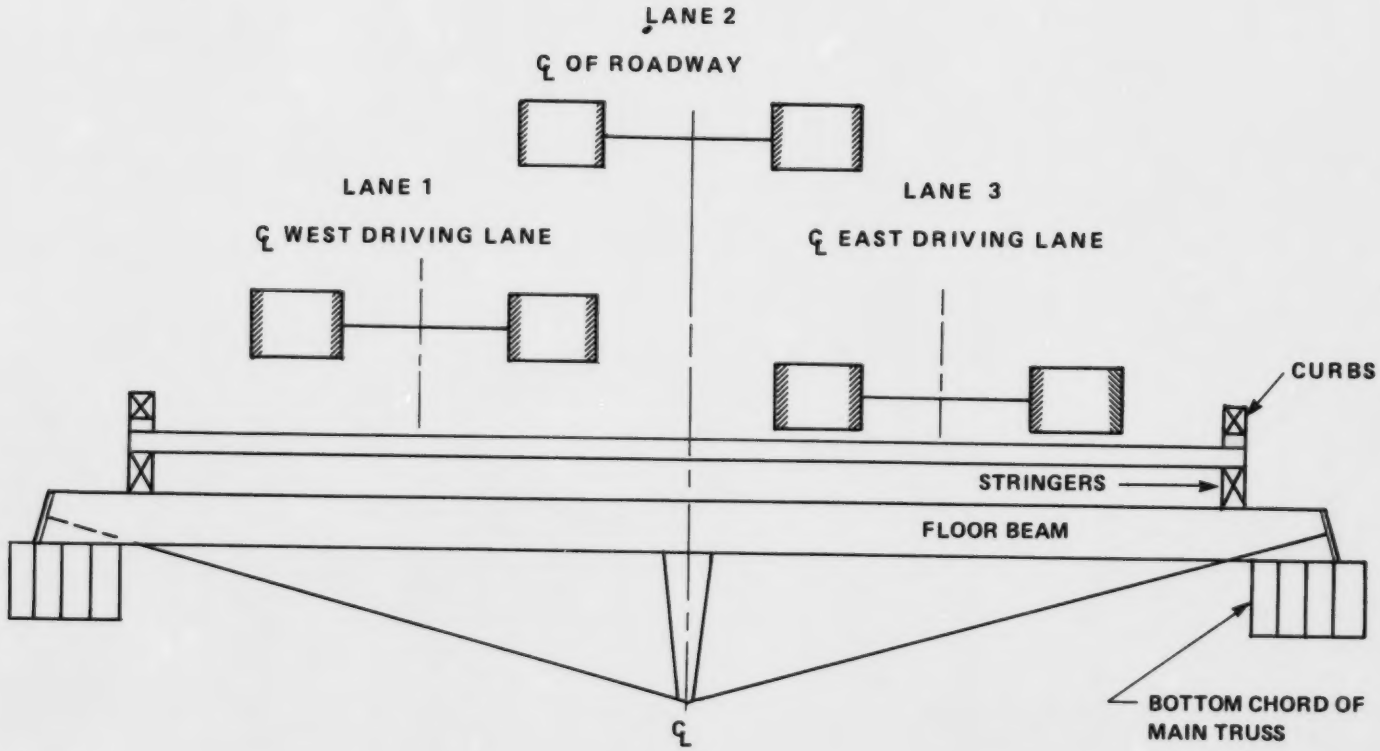
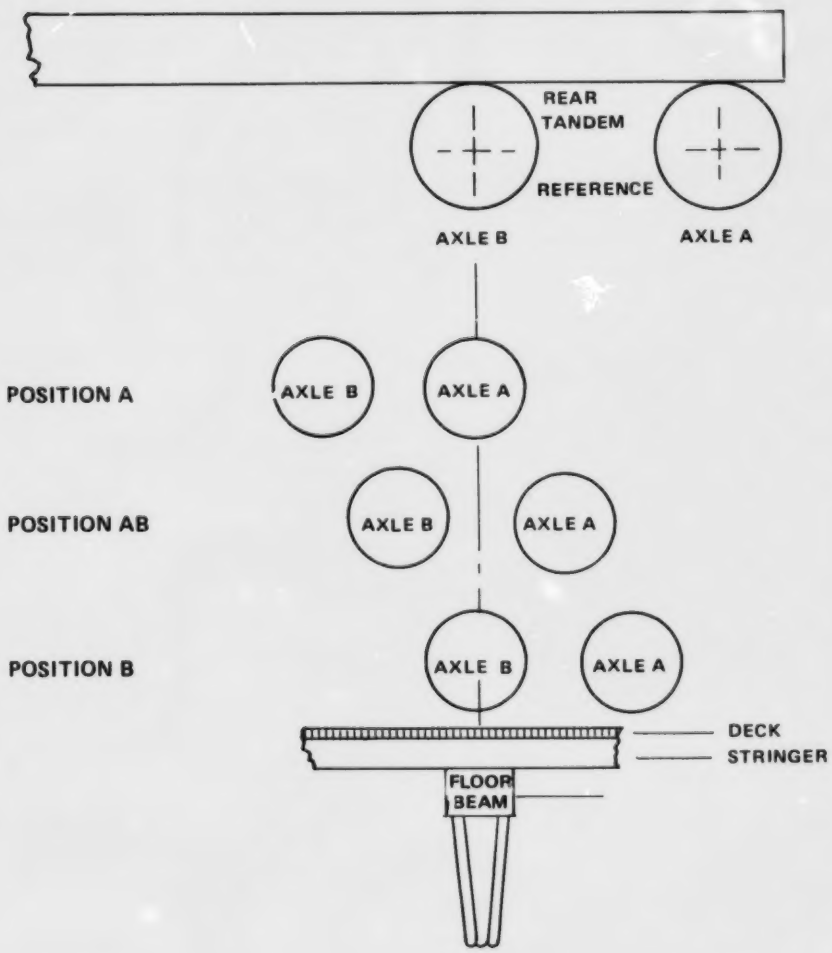


Figure 18/ Transverse Vehicle Positions, Preliminary Testing Main Span Deck



- Positions A, AB & B for first 5 floor beams from south end for lanes 1 & 2 (see Figure 18)
- Position A for first 5 floor beams from south end for lane 3

Figure 19/ Longitudinal Vehicle Placement, Preliminary Testing Main Span Deck

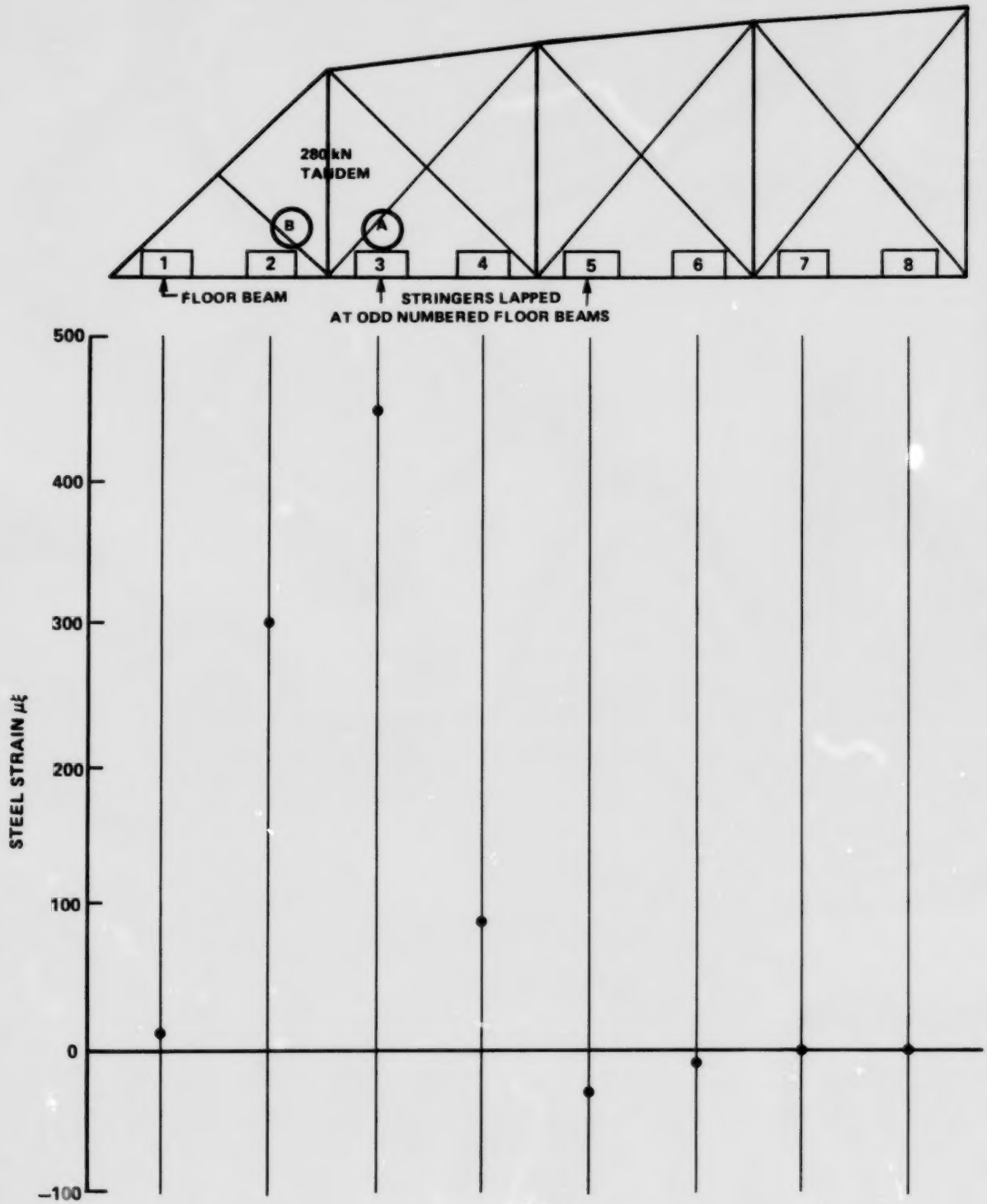


Figure 20/ Steel Strains in Floor Beams, Lane Position 2

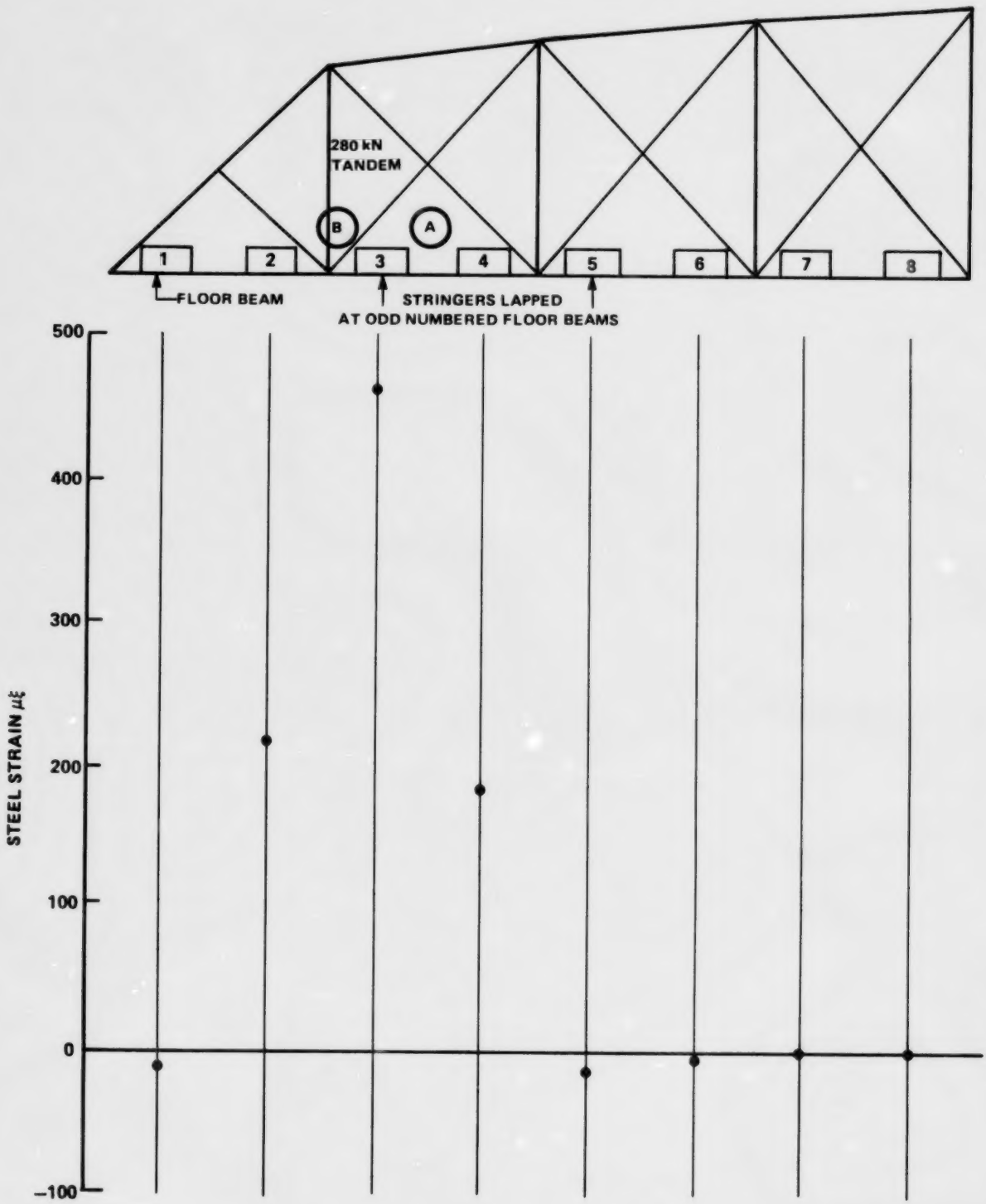
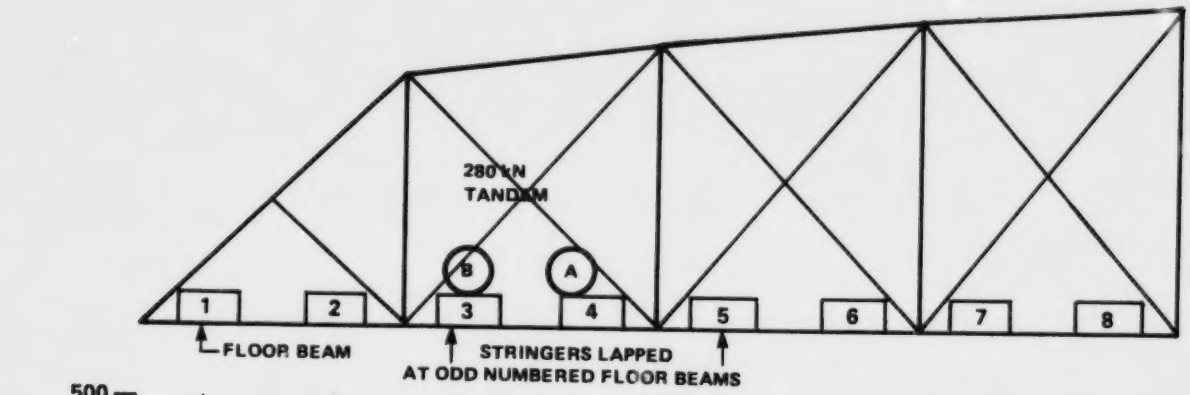


Figure 21/ Steel Strains in Floor Beams, Lane Position 2



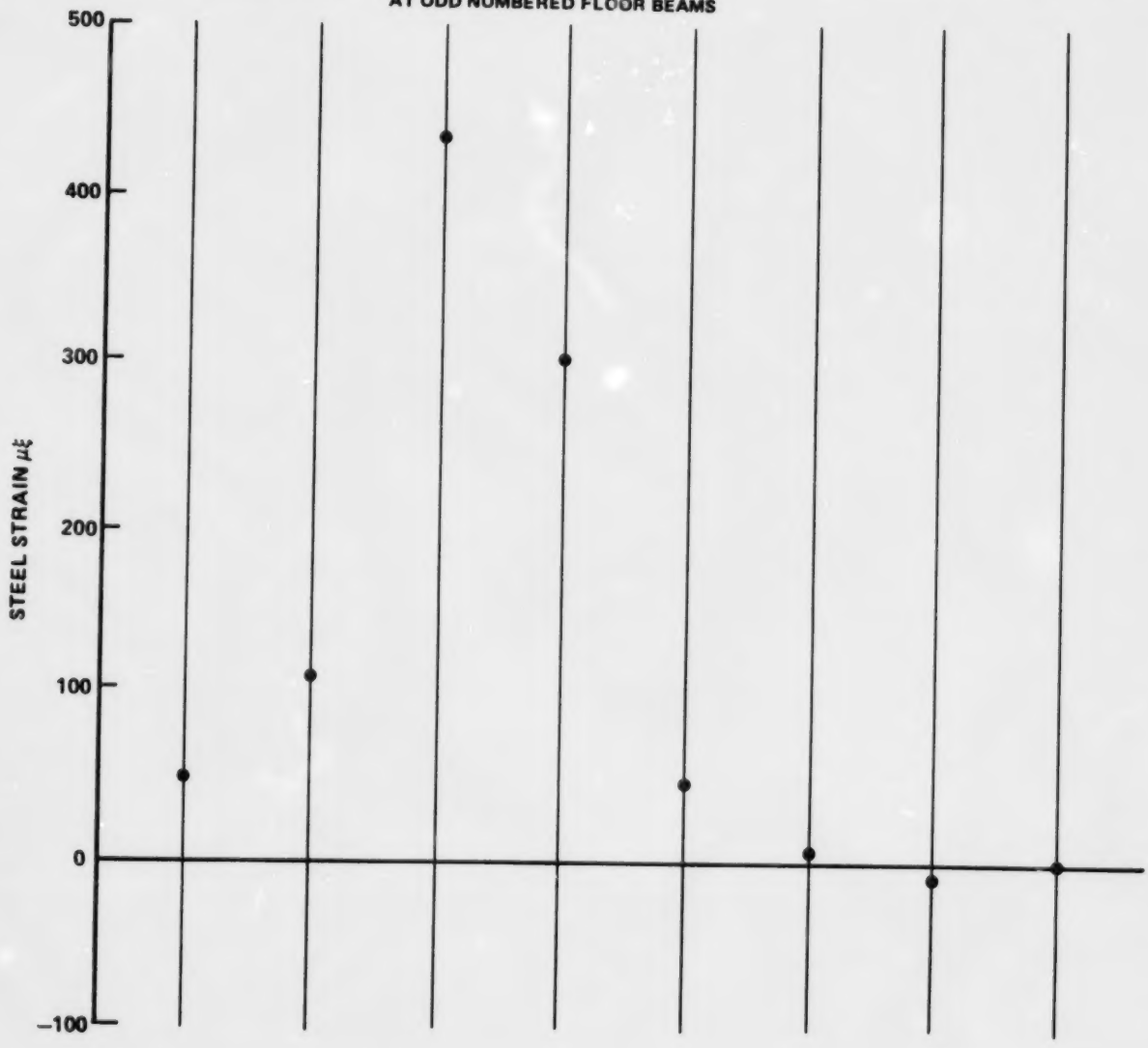


Figure 22/ Steel Strains in Floor Beams, Lane Position 2

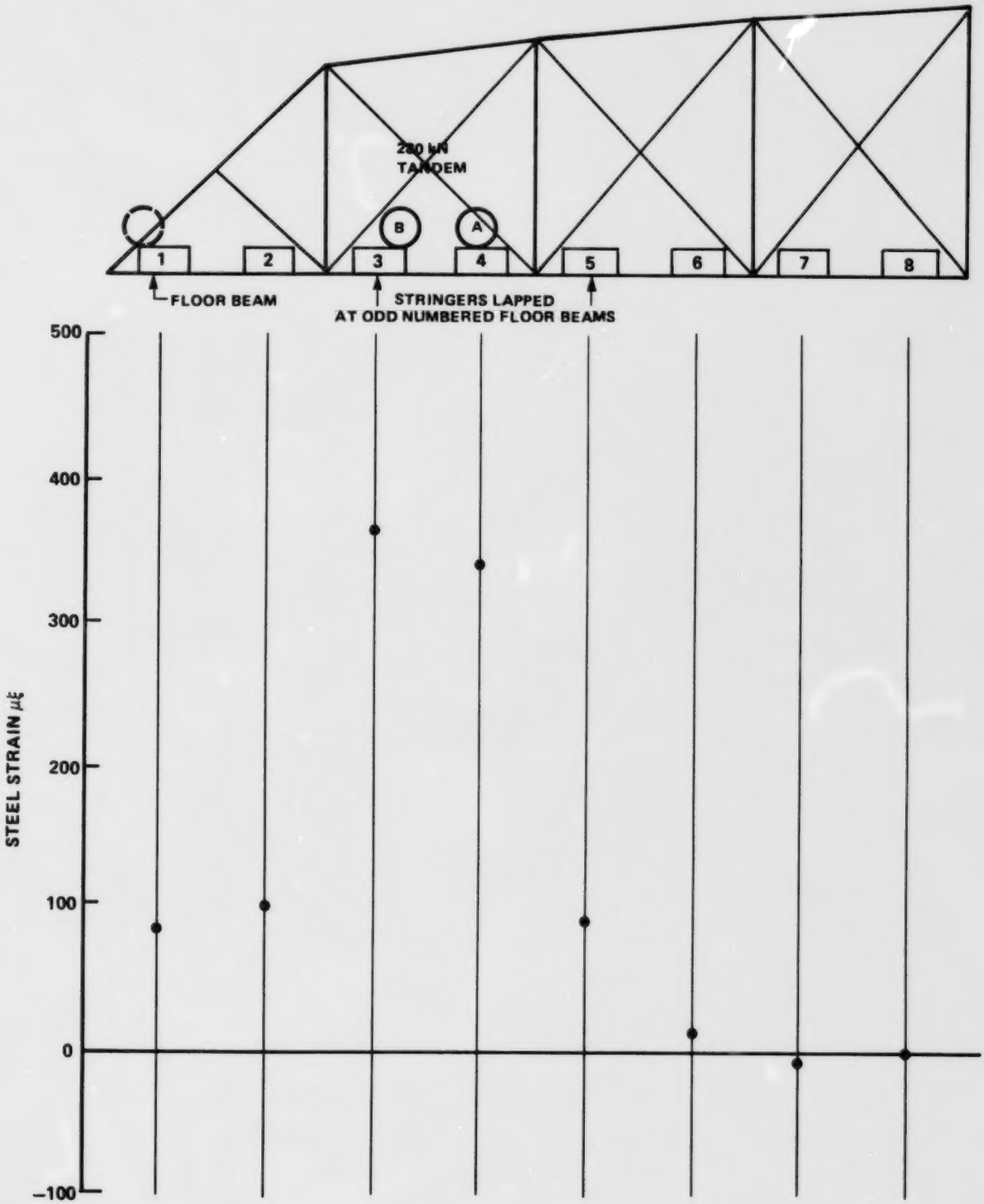


Figure 23/ Steel Strains in Floor Beams, Lane Position 2

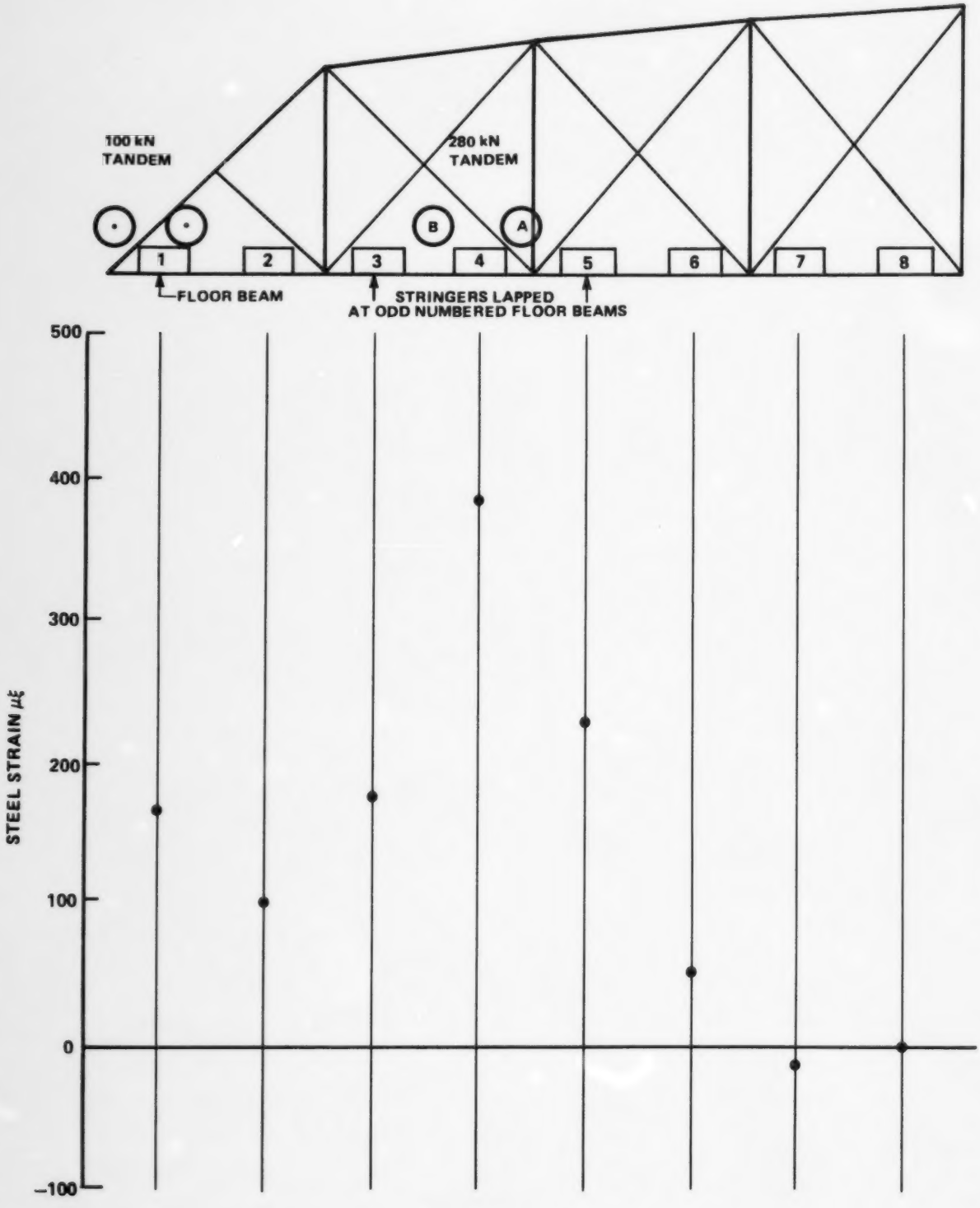


Figure 24/ Steel Strains in Floor Beams, Lane Position 2

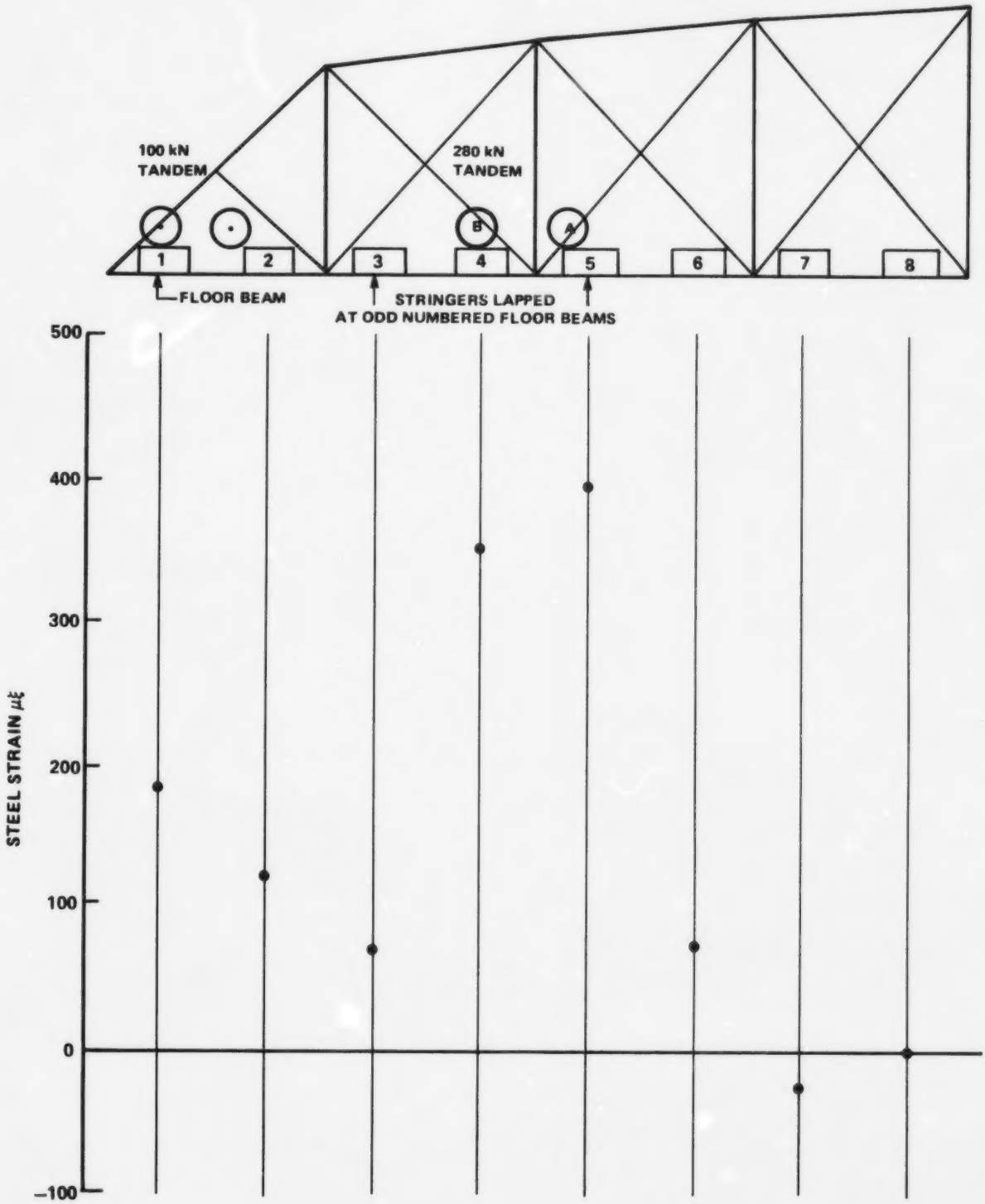


Figure 25/ Steel Strains in Floor Beams, Lane Position 2

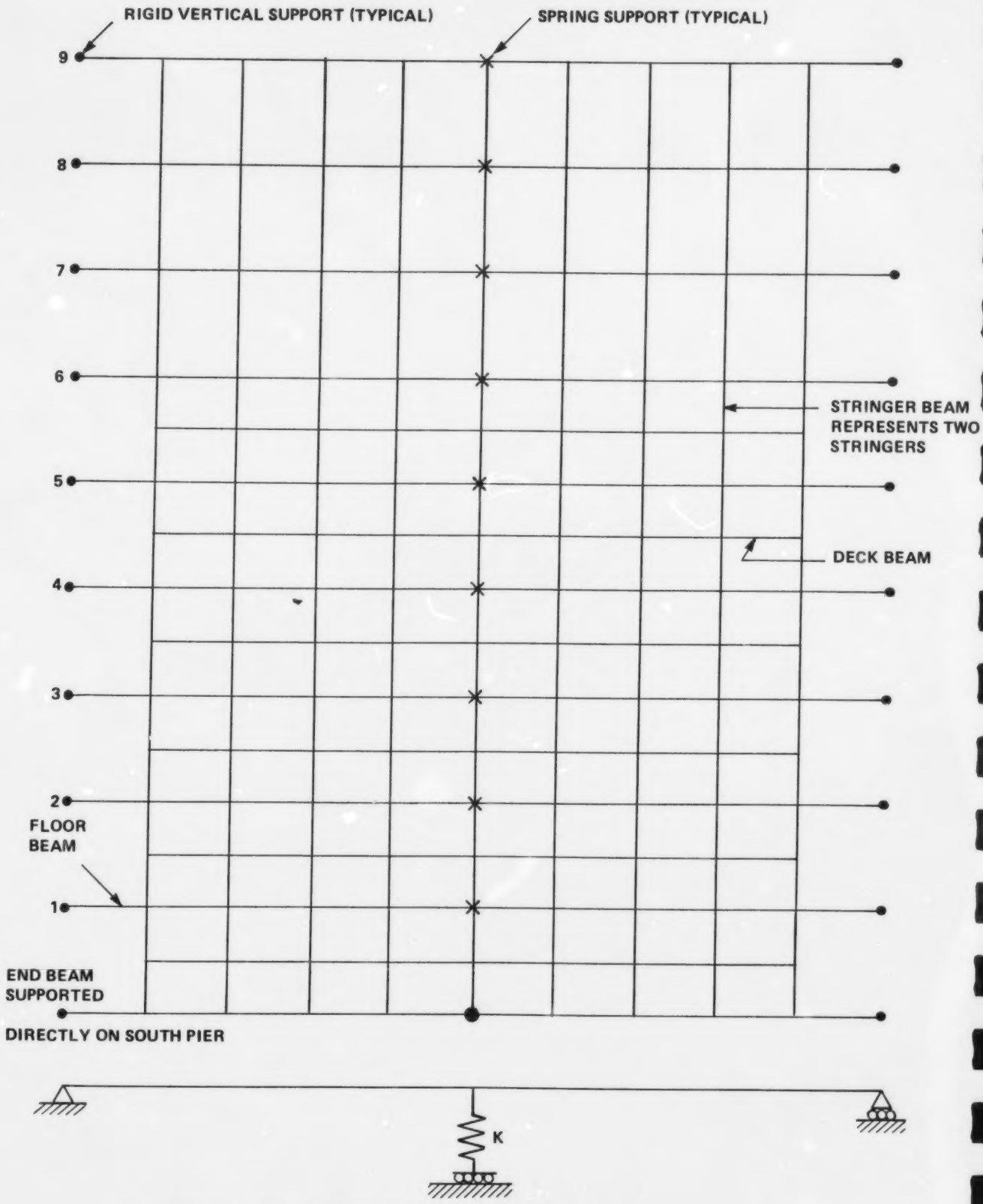


Figure 26/ Deck Grid Analogy, Main Span Deck System

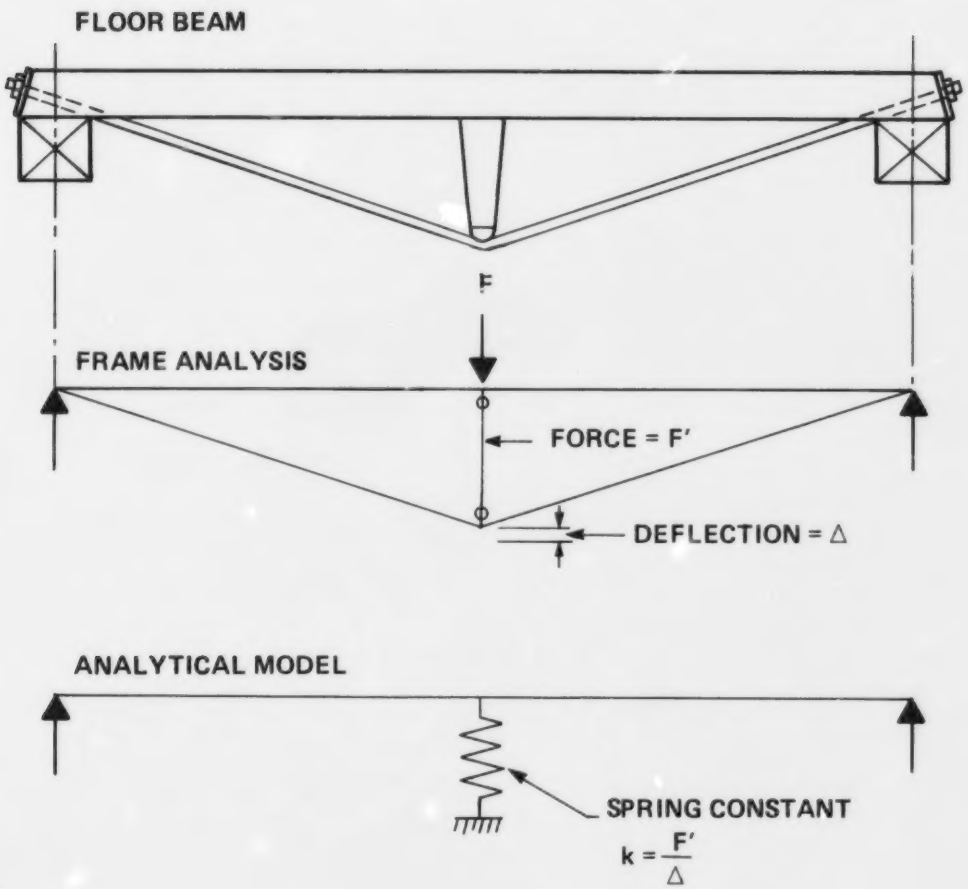


Figure 27/ Simulation of Kingpost by Spring Support

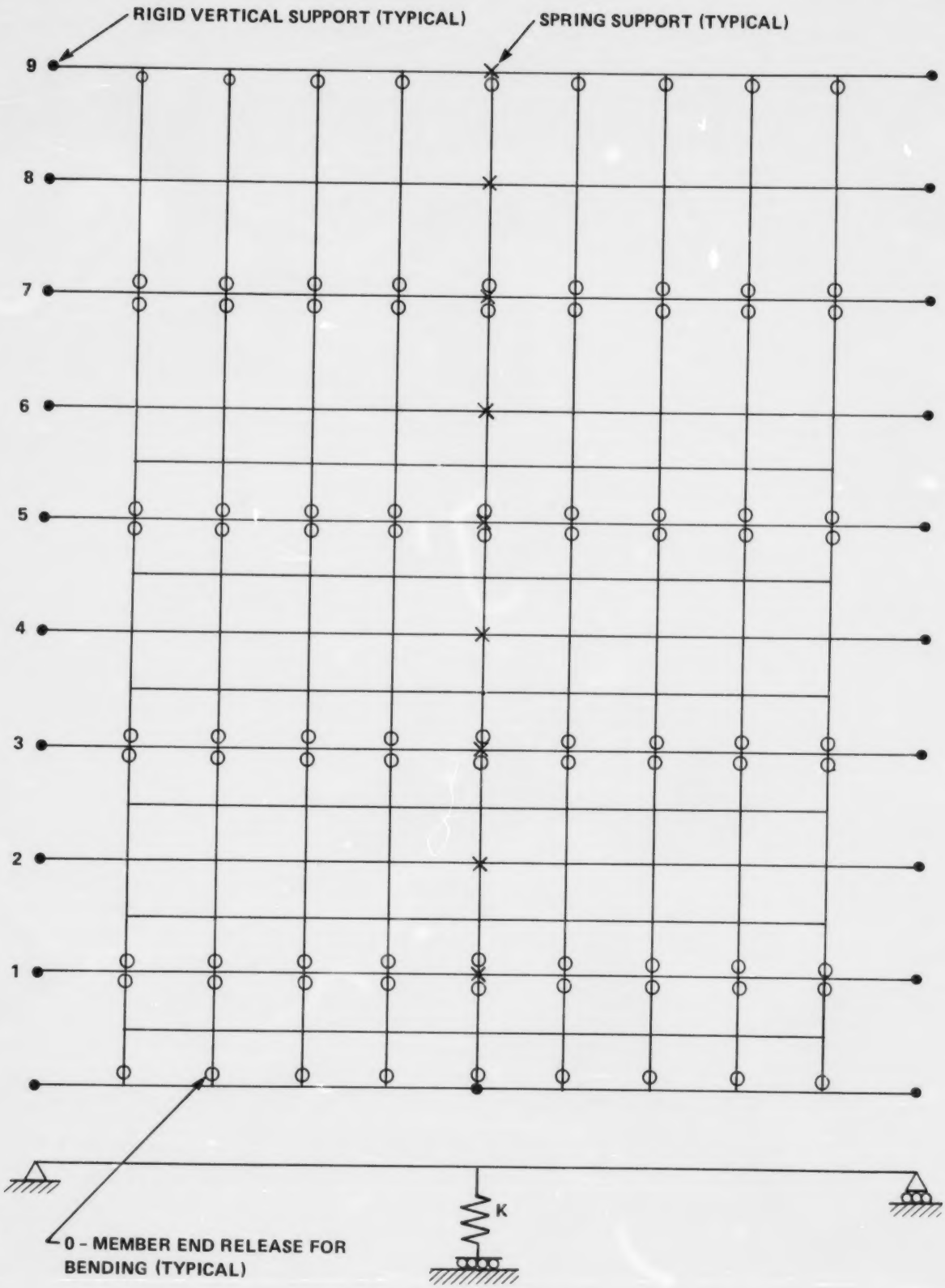


Figure 28/ Deck Grid Analogy, Grid 1

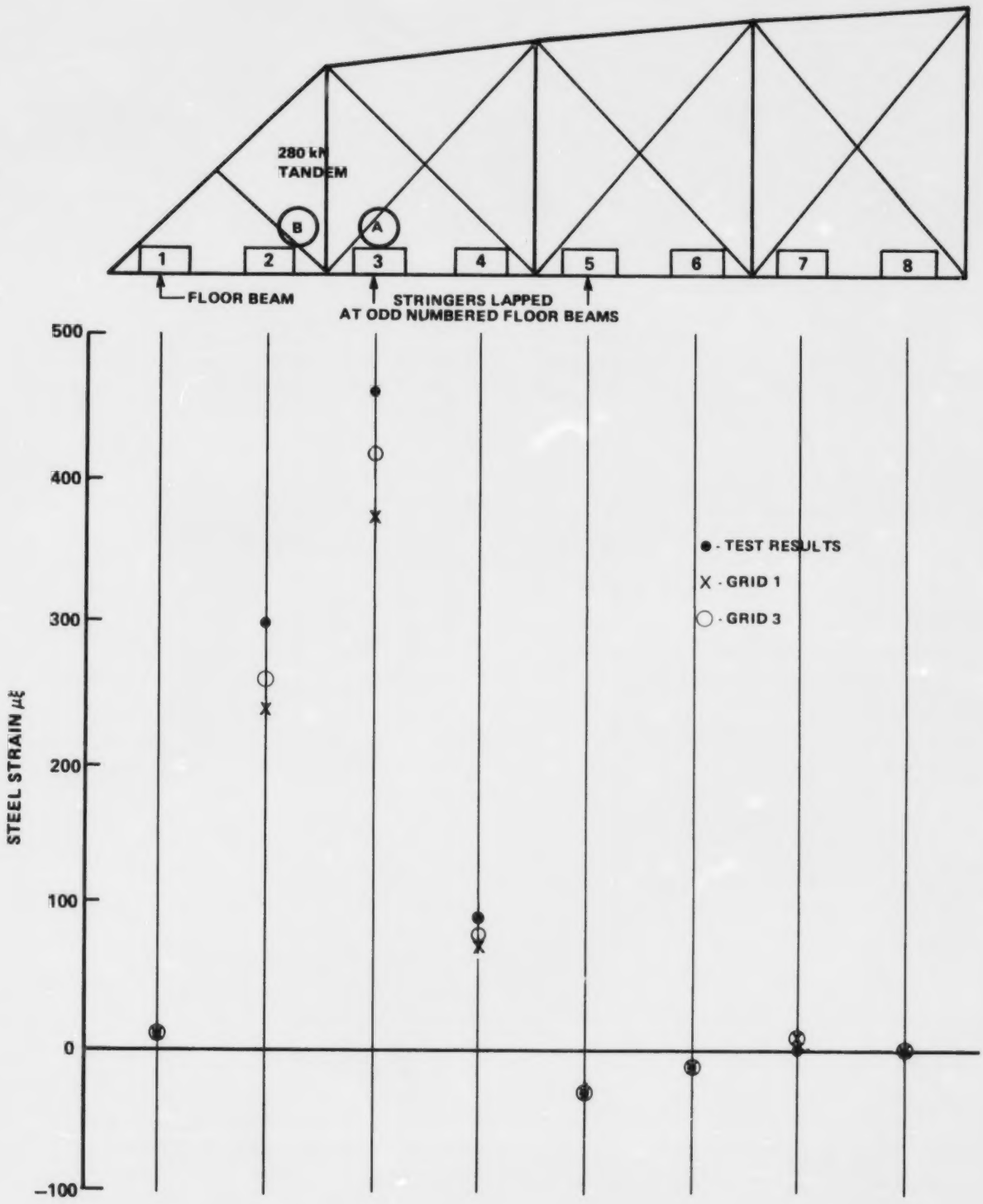


Figure 29/ Analytical vs Experimental Floor Beam Strains

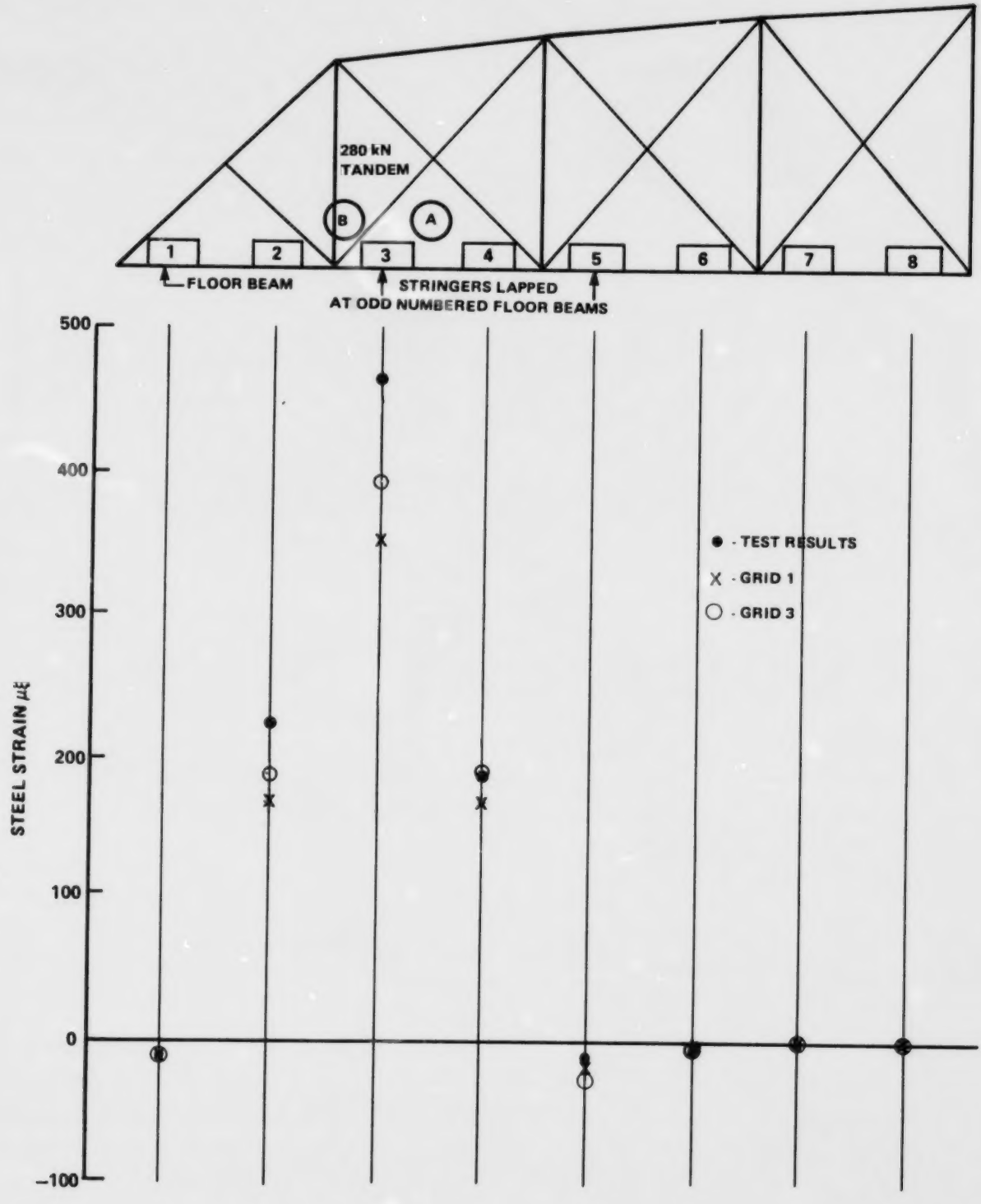


Figure 30/ Analytical vs Experimental Floor Beam Strains

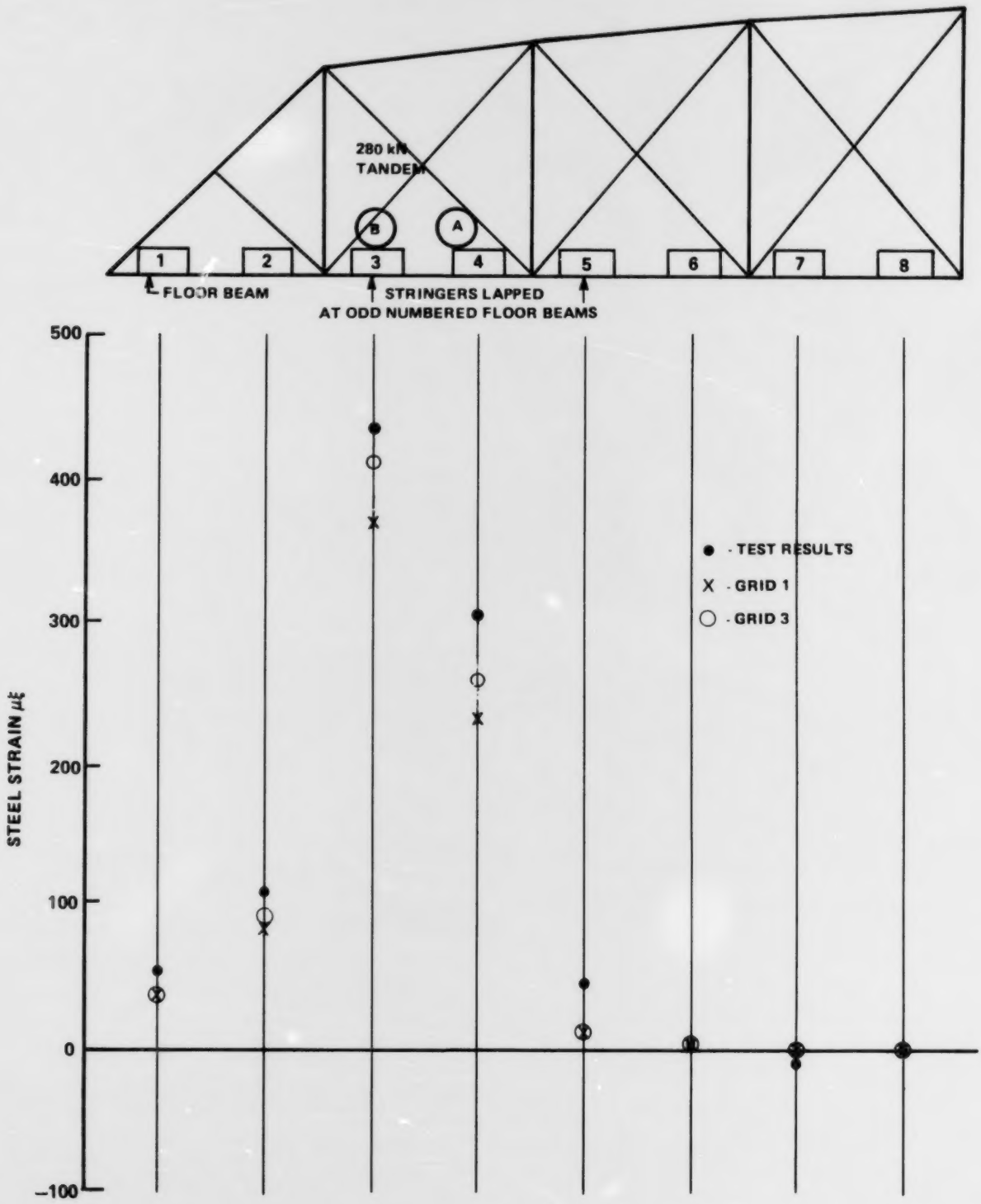


Figure 31/ Analytical vs Experimental Floor Beam Strains

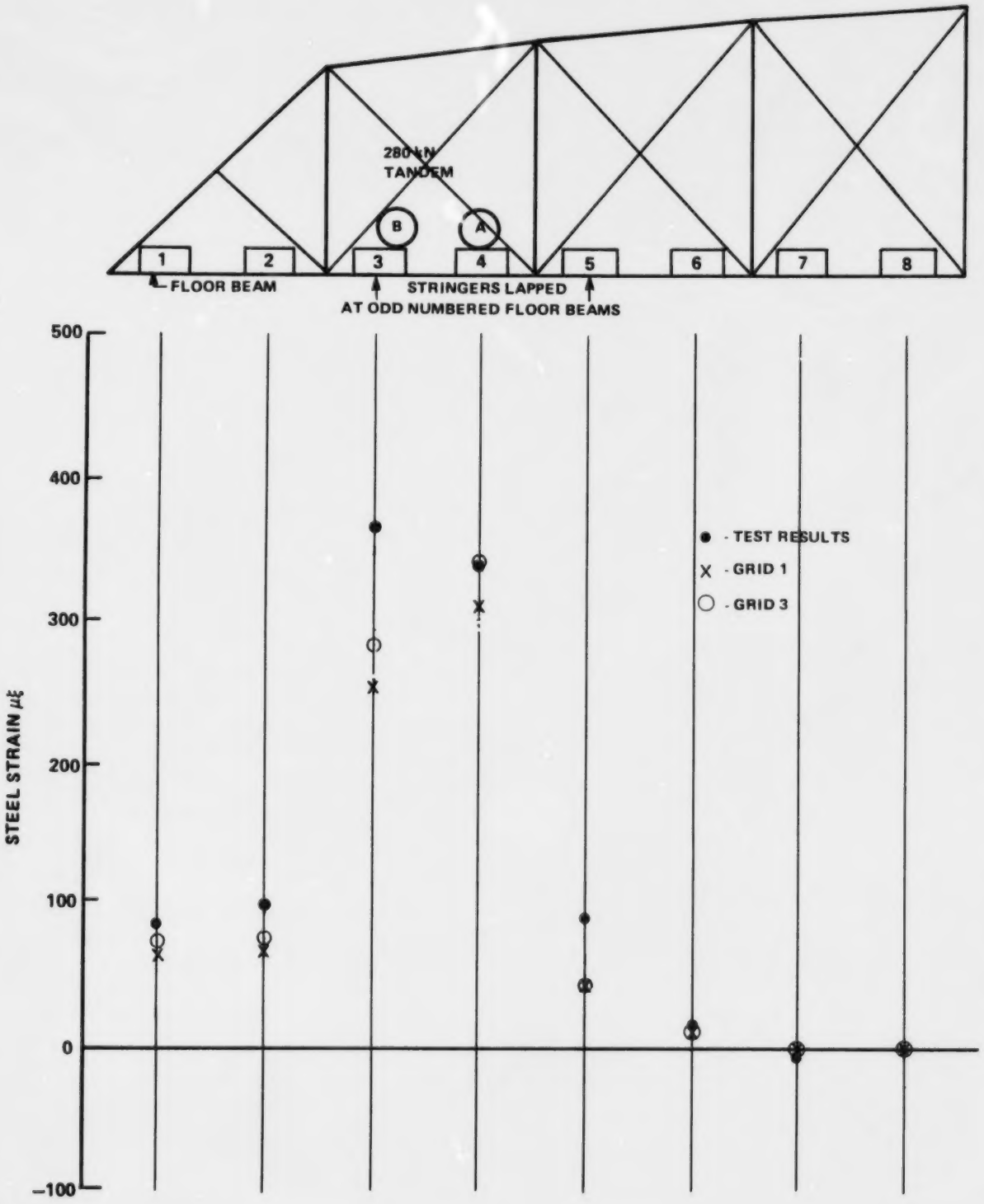


Figure 32/ Analytical vs Experimental Floor Beam Strains

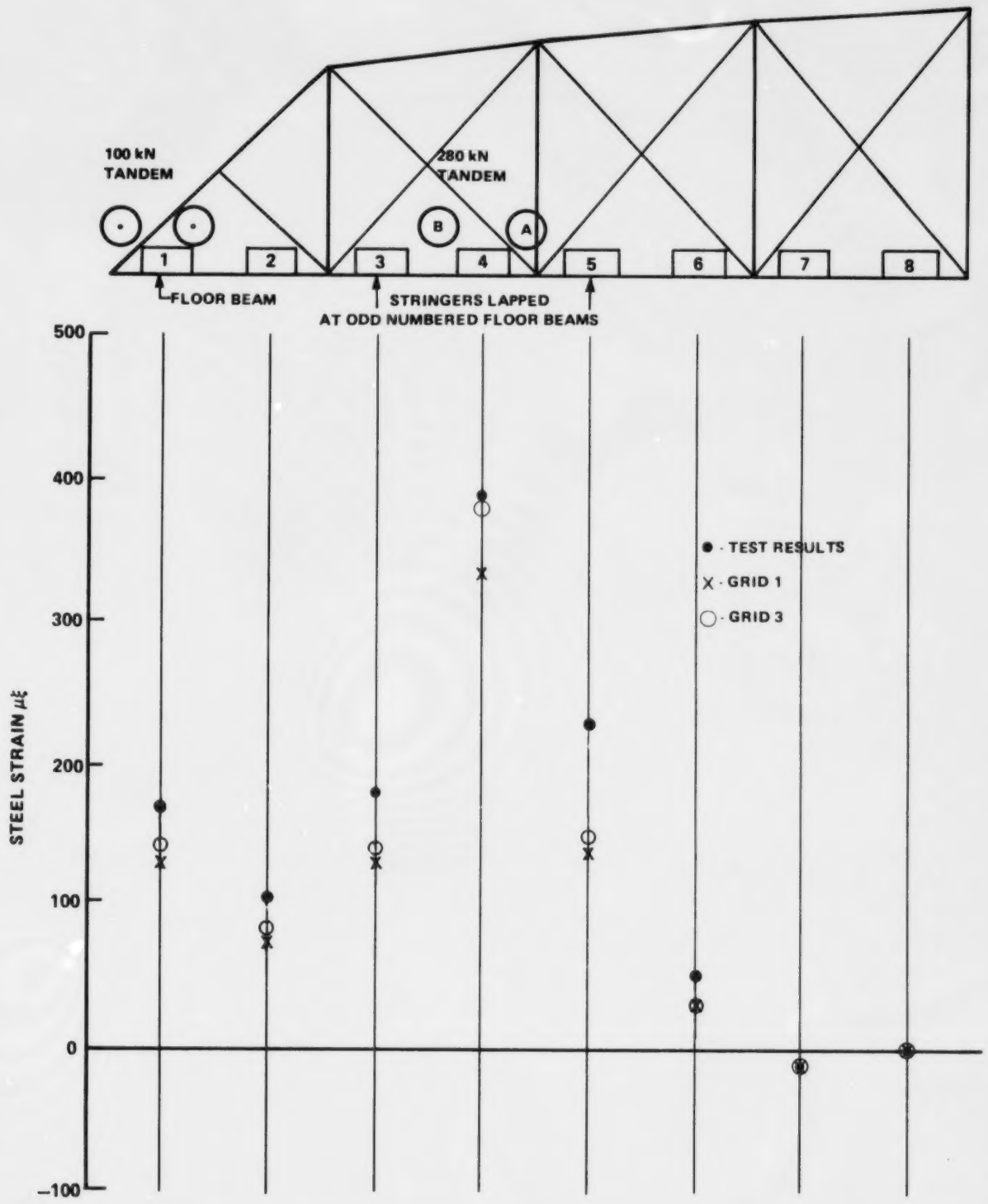


Figure 33/ Analytical vs Experimental Floor Beam Strains

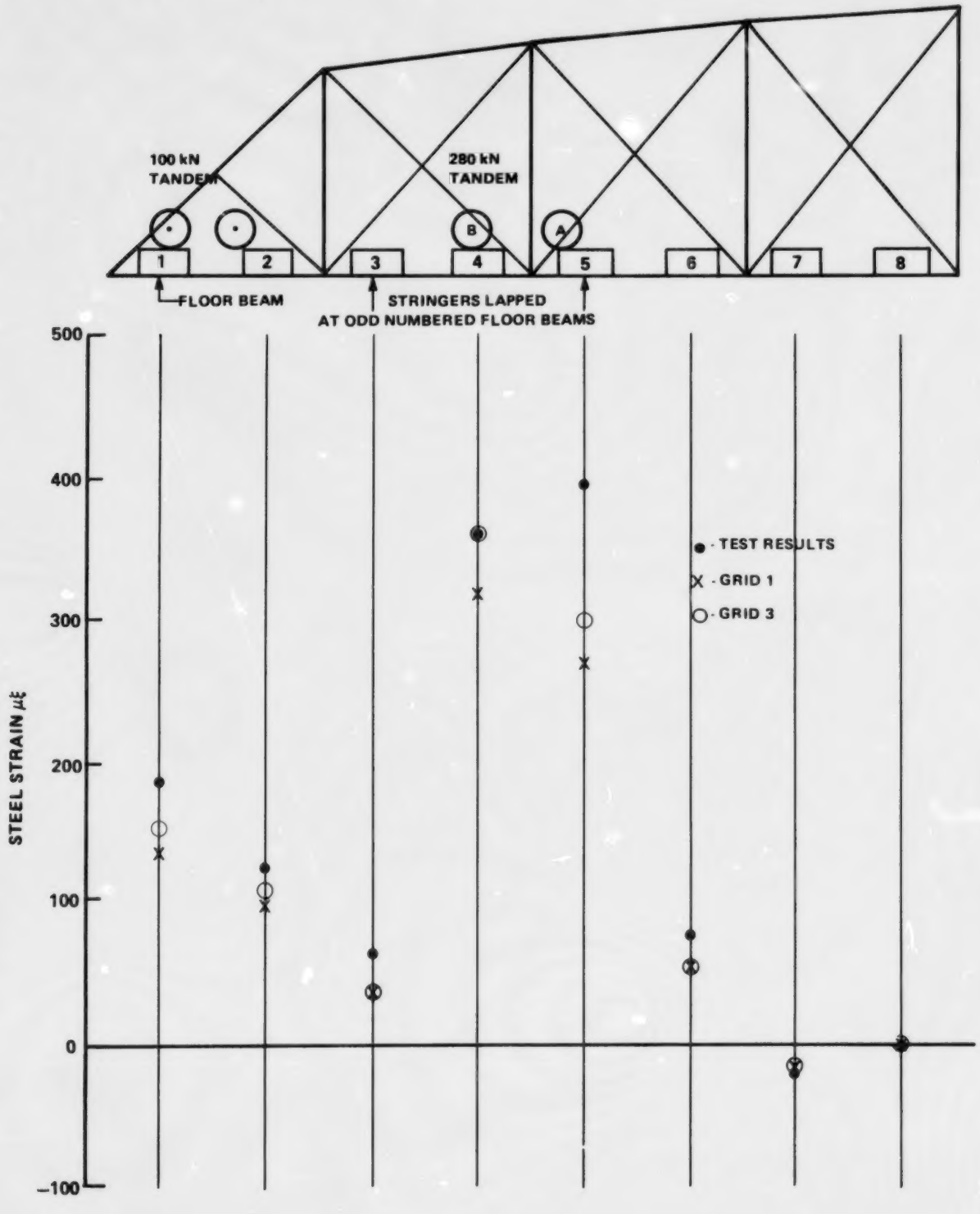


Figure 34/ Analytical vs Experimental Floor Beam Strains

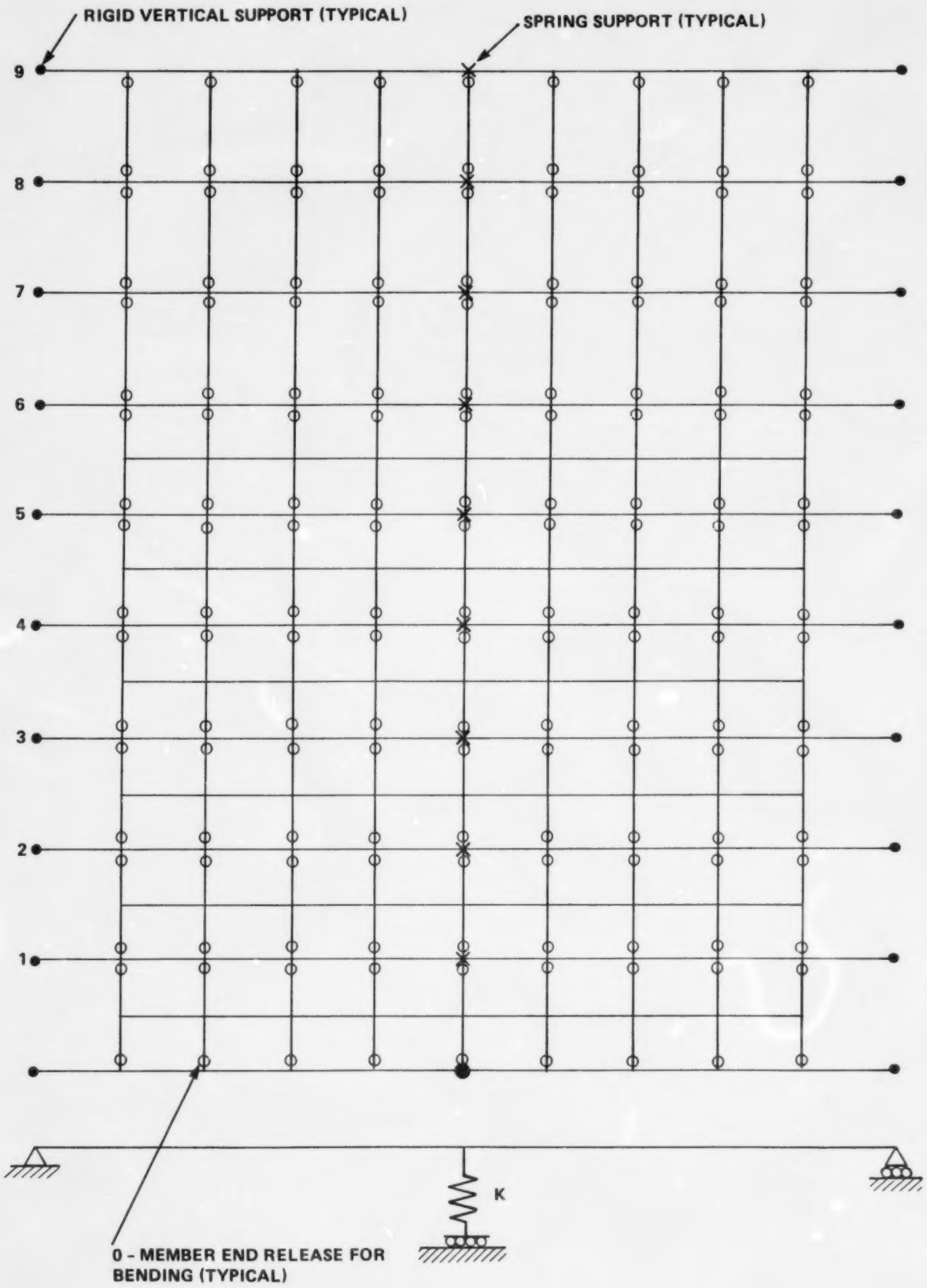


Figure 35/ Deck Grid Analogy, Grid 4

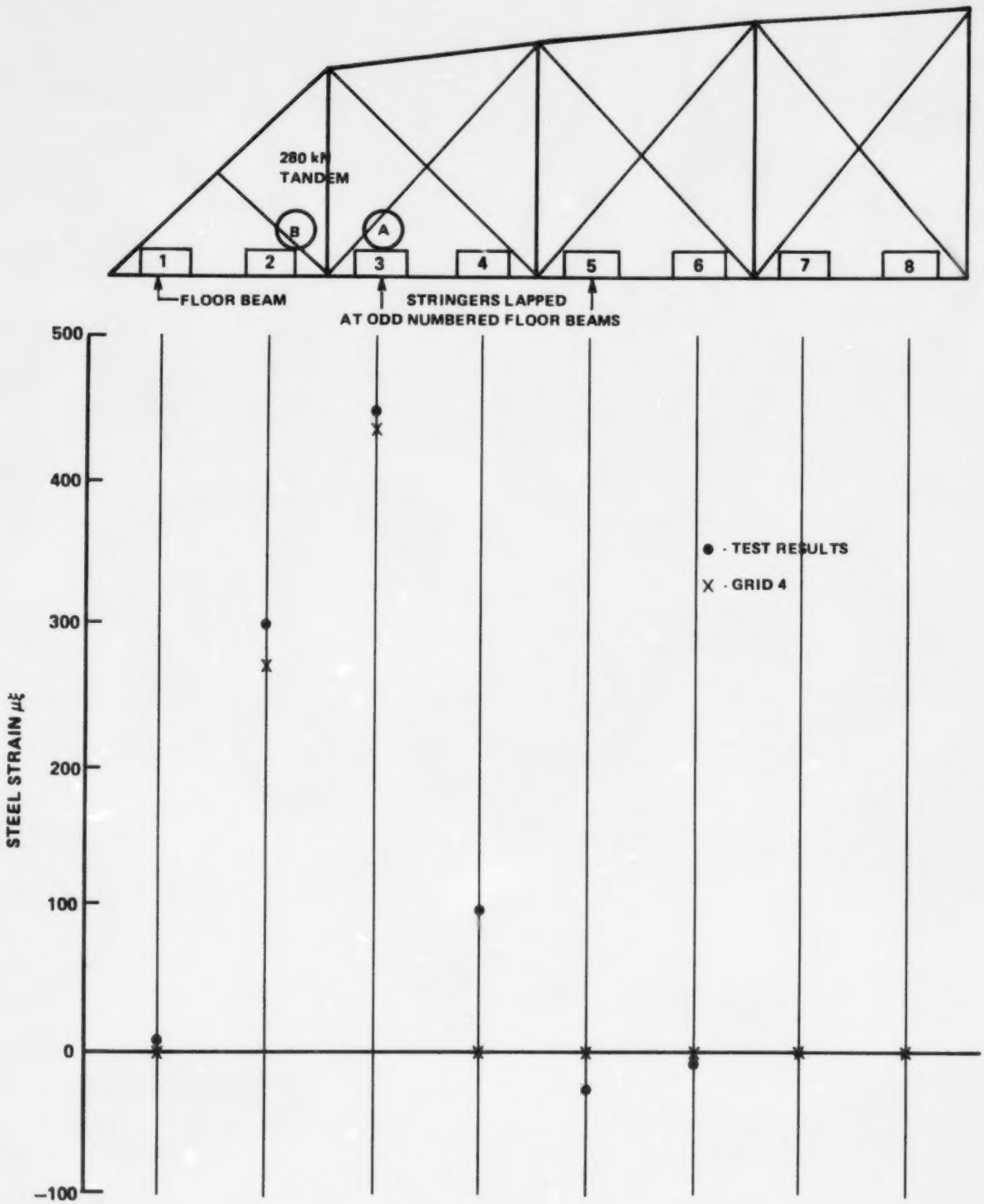


Figure 36/ Analytical vs Experimental Floor Beam Strains

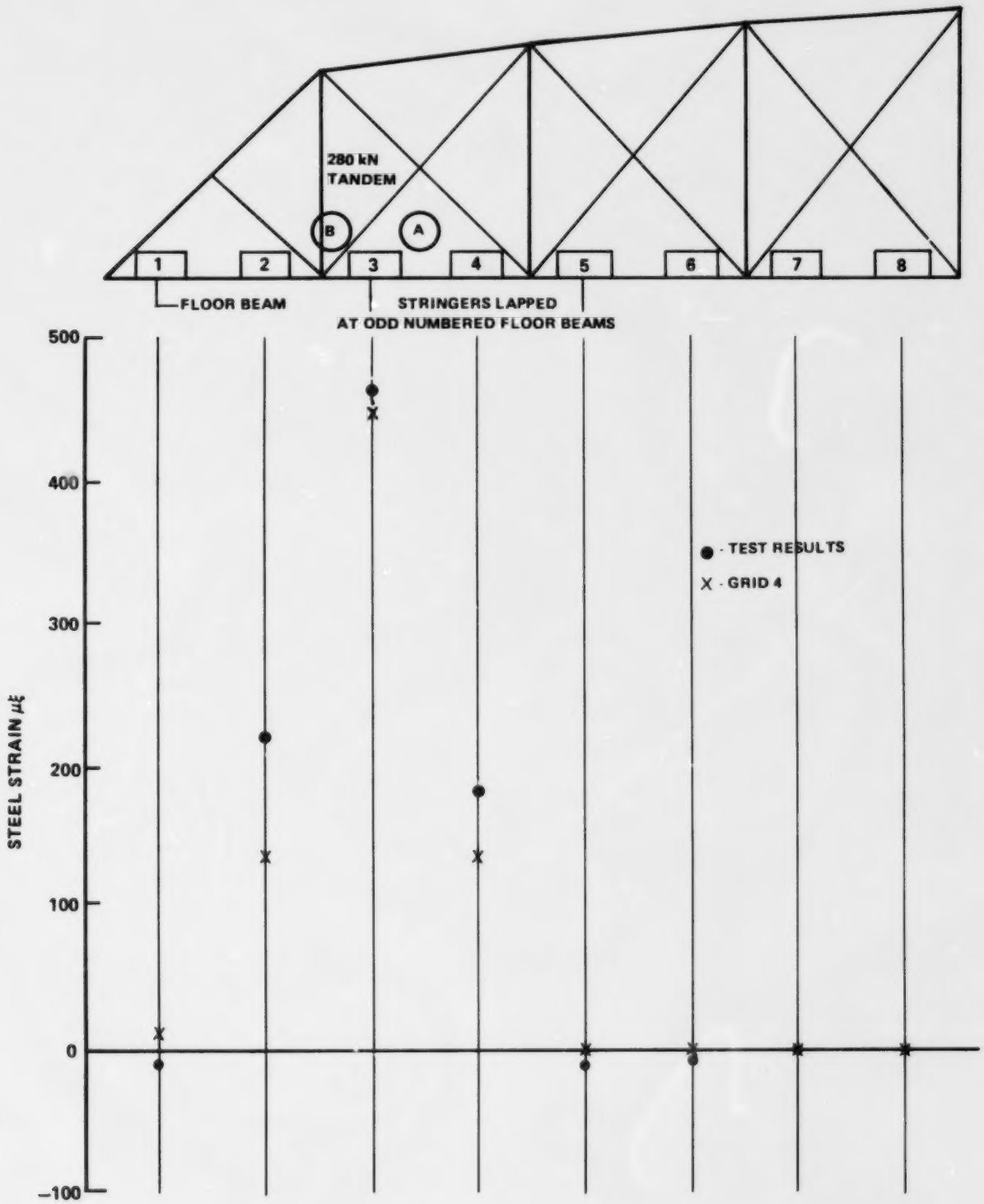


Figure 37/ Analytical vs Experimental Floor Beam Strains

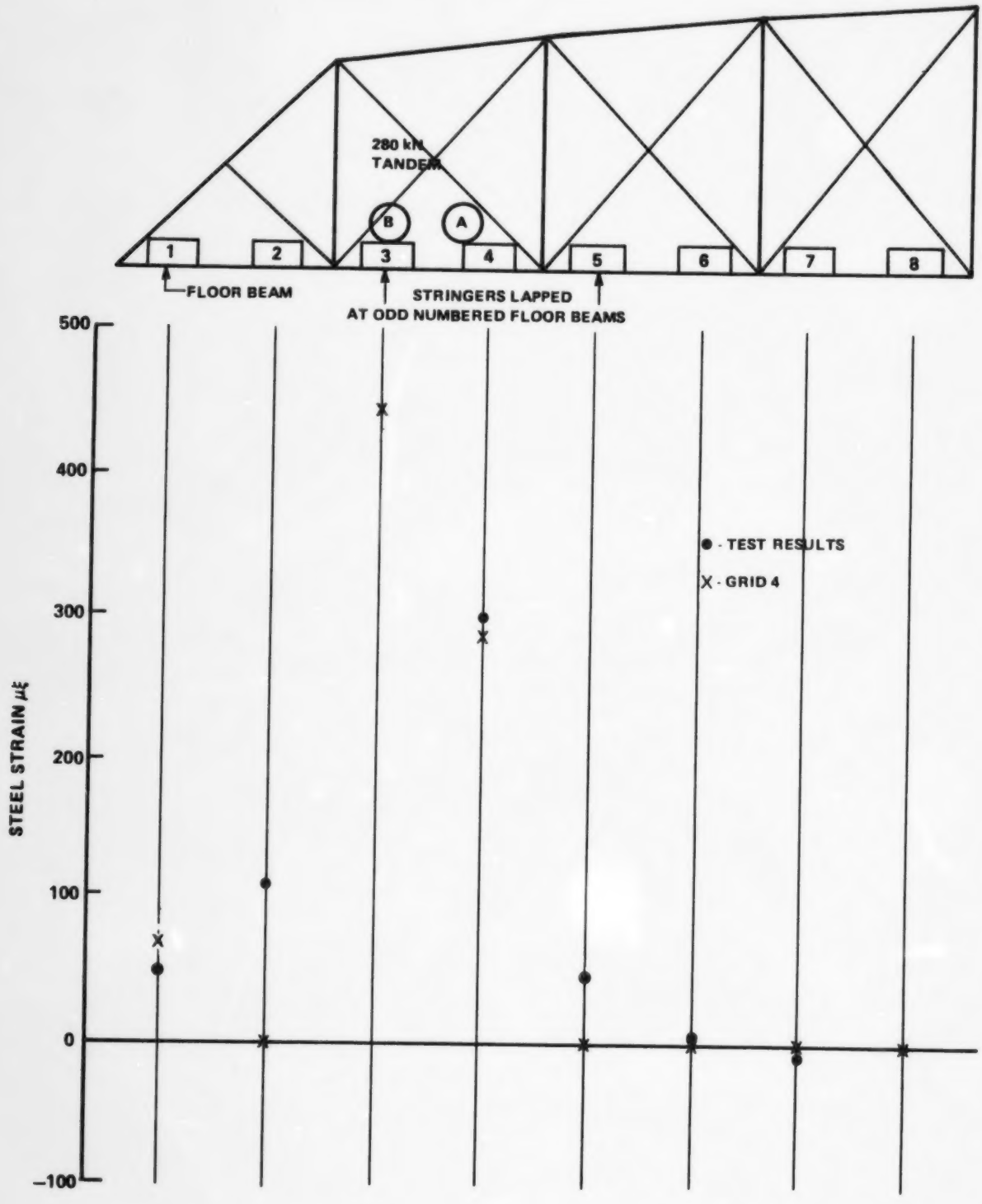


Figure 38/ Analytical vs Experimental Floor Beam Strains

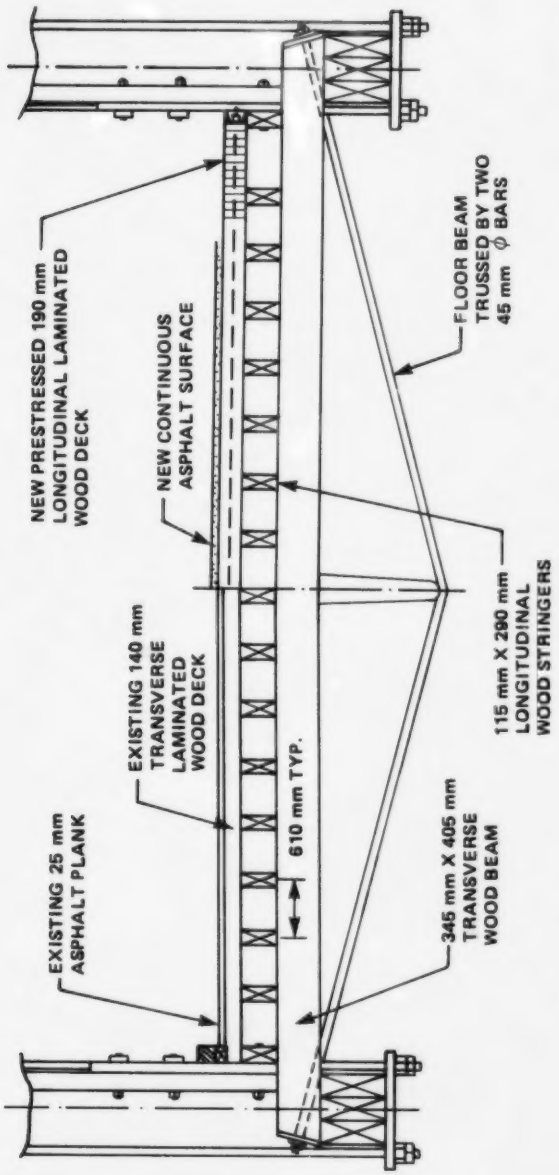


Figure 39/ New Longitudinal Laminated Deck

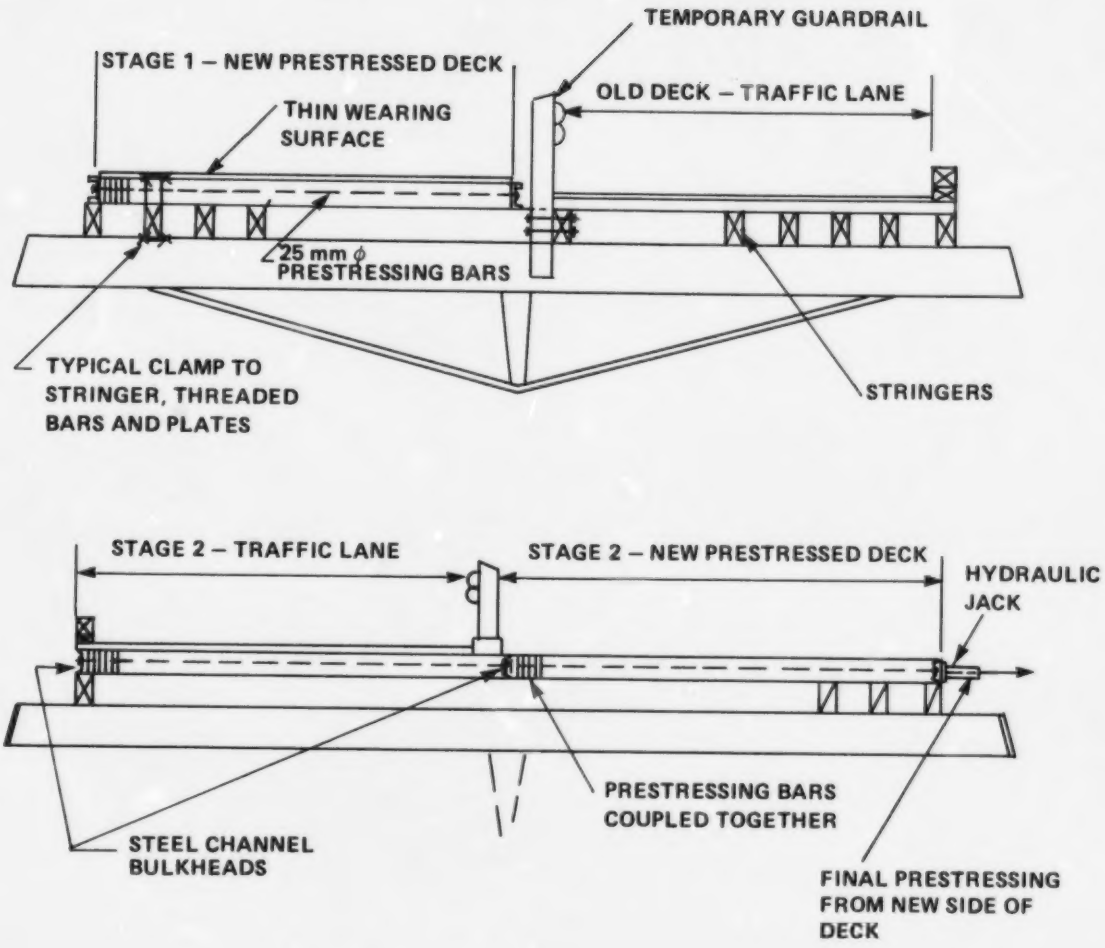

TEMPORARY GUARDRAIL
STAGE 1 – NEW PRESTRESSED DECK
OLD DECK – TRAFFIC LANE
THIN WEARING SURFACE
25 mm ϕ PRESTRESSING BARS
TYPICAL CLAMP TO STRINGER, THREADED BARS AND PLATES
STRINGERS
STAGE 2 – TRAFFIC LANE
STAGE 2 – NEW PRESTRESSED DECK
HYDRAULIC JACK
PRESTRESSING BARS COUPLED TOGETHER
STEEL CHANNEL BULKHEADS
FINAL PRESTRESSING FROM NEW SIDE OF DECK

Figure 40/ Two-Stage Construction of New Deck

Figure 41/ Deck Rehabilitation, New Half Deck Replacement

Figure 42/ Deck Rehabilitation, Second Half Deck Replacement

Figure 43/ Completed New Deck with Asphalt Surface

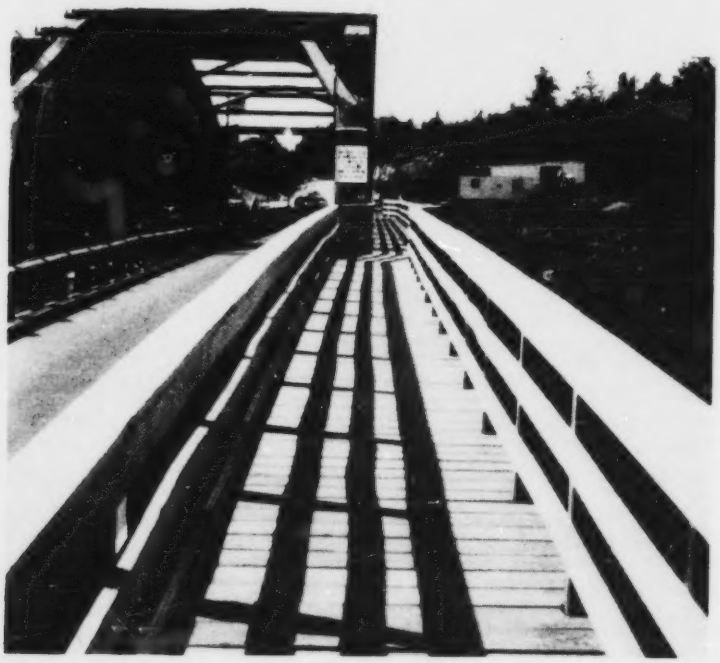


Figure 44/ New Wood Outrigger Sidewalk

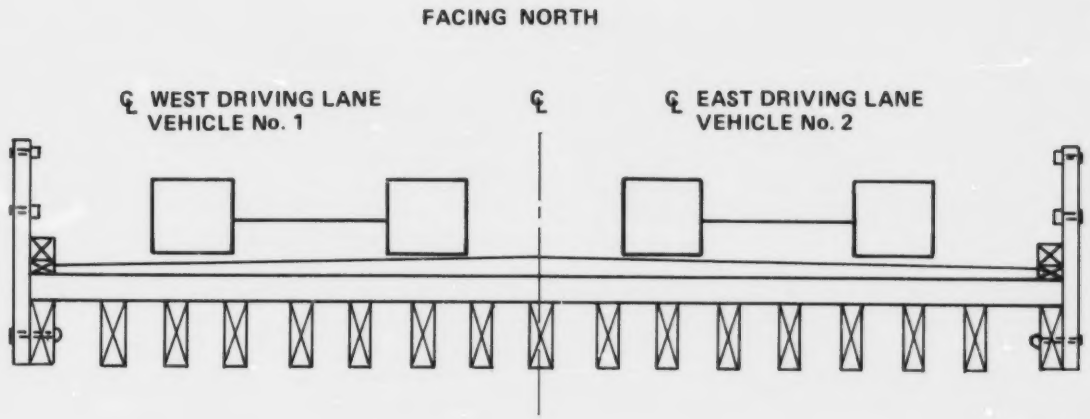


Figure 45/ Transverse Vehicle Positions, Proof Testing Two Vehicles

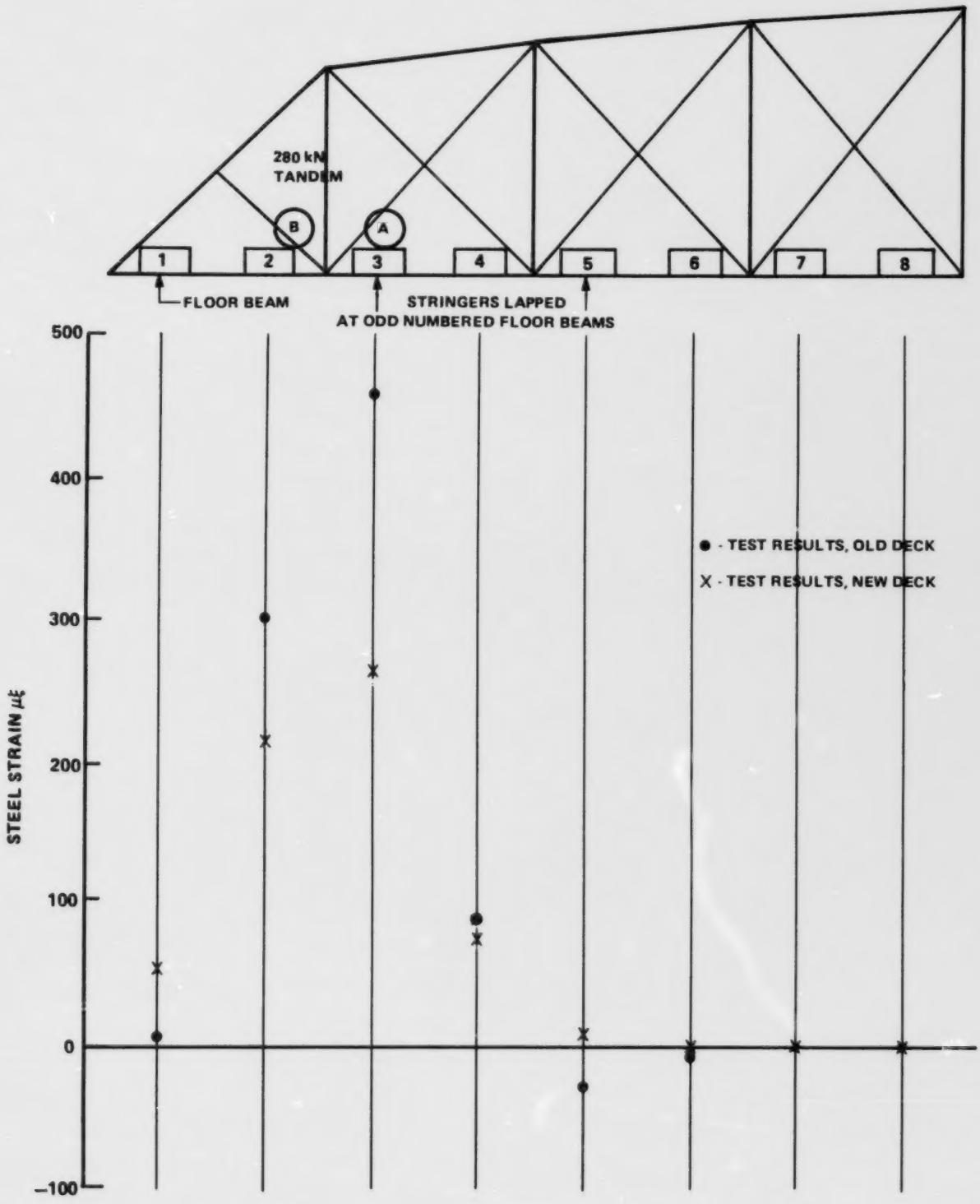


Figure 46/ Steel Strains in Floor Beams, Lane Position 2

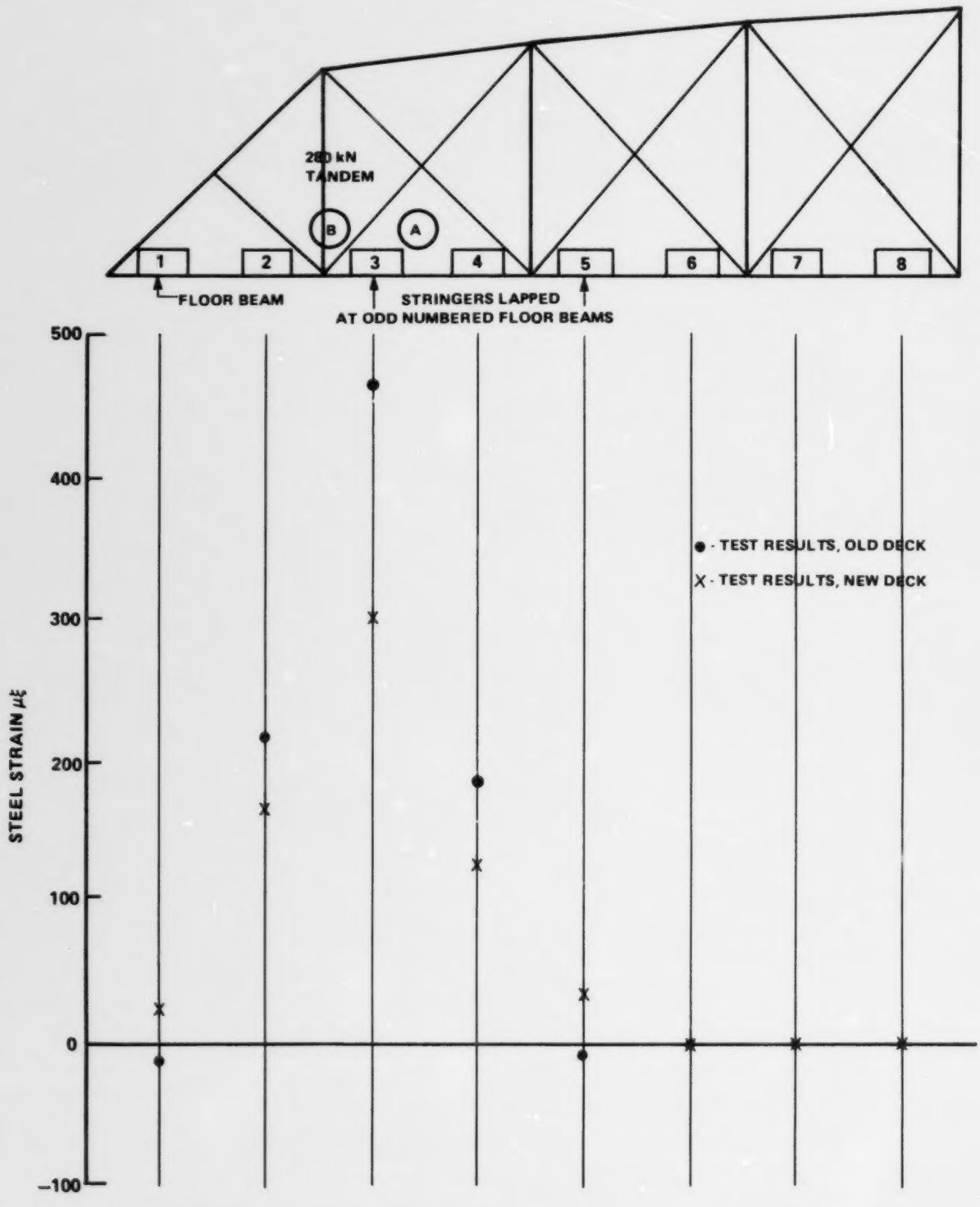


Figure 47/ Steel Strains in Floor Beams, Lane Position 2

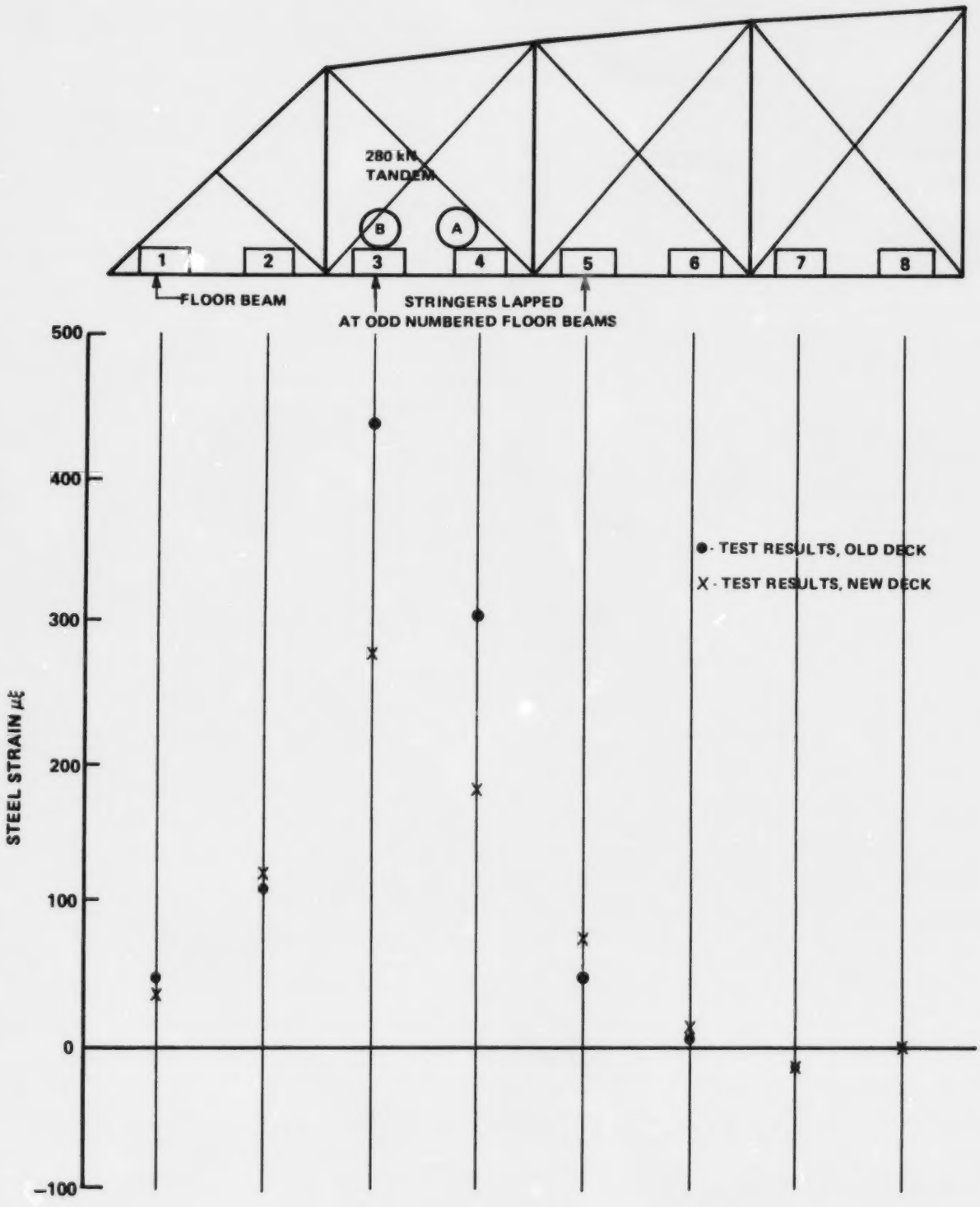


Figure 48/ Steel Strains in Floor Beams, Lane Position2

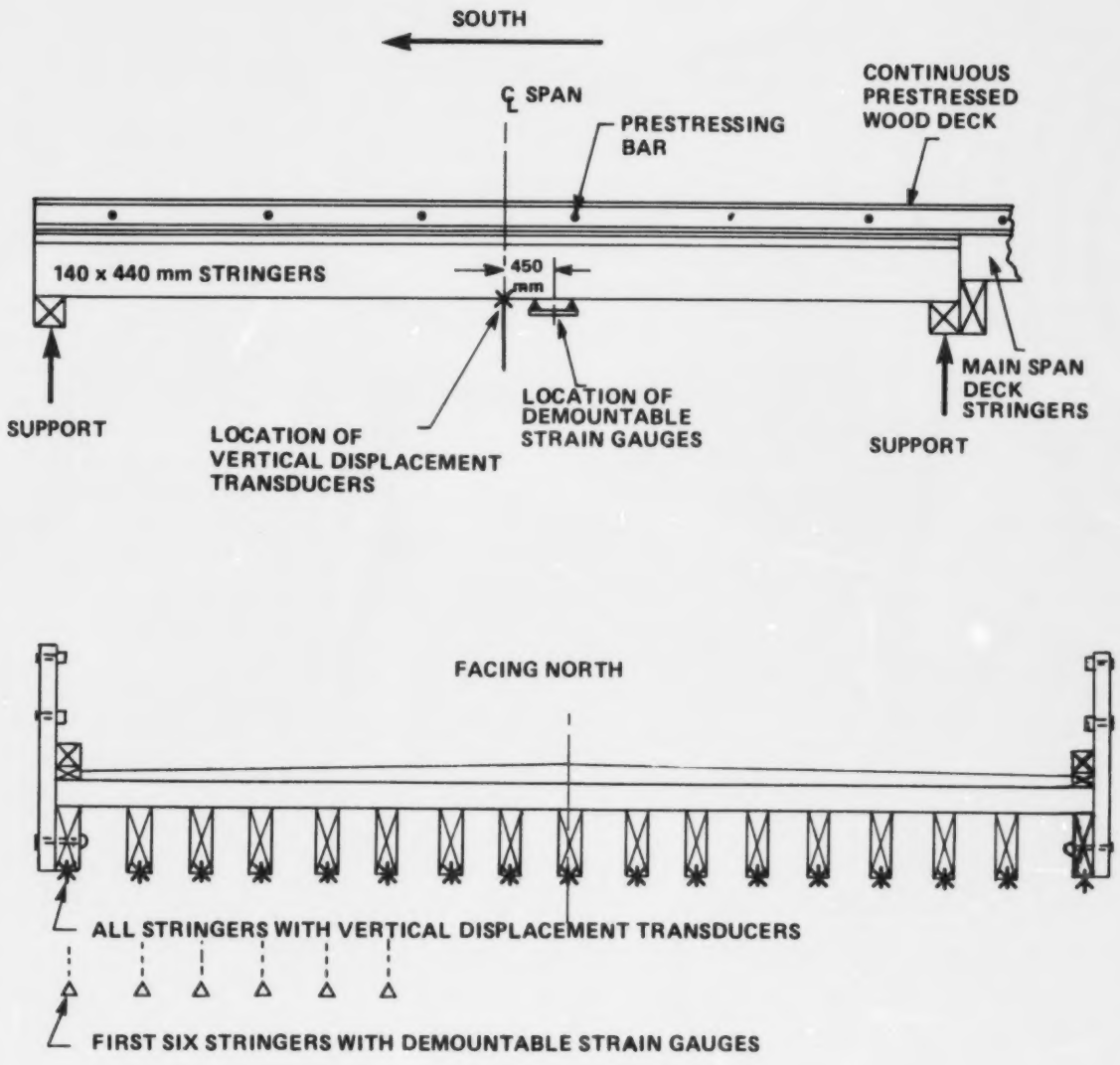


Figure 49/ Instrumentation, South Approach Span

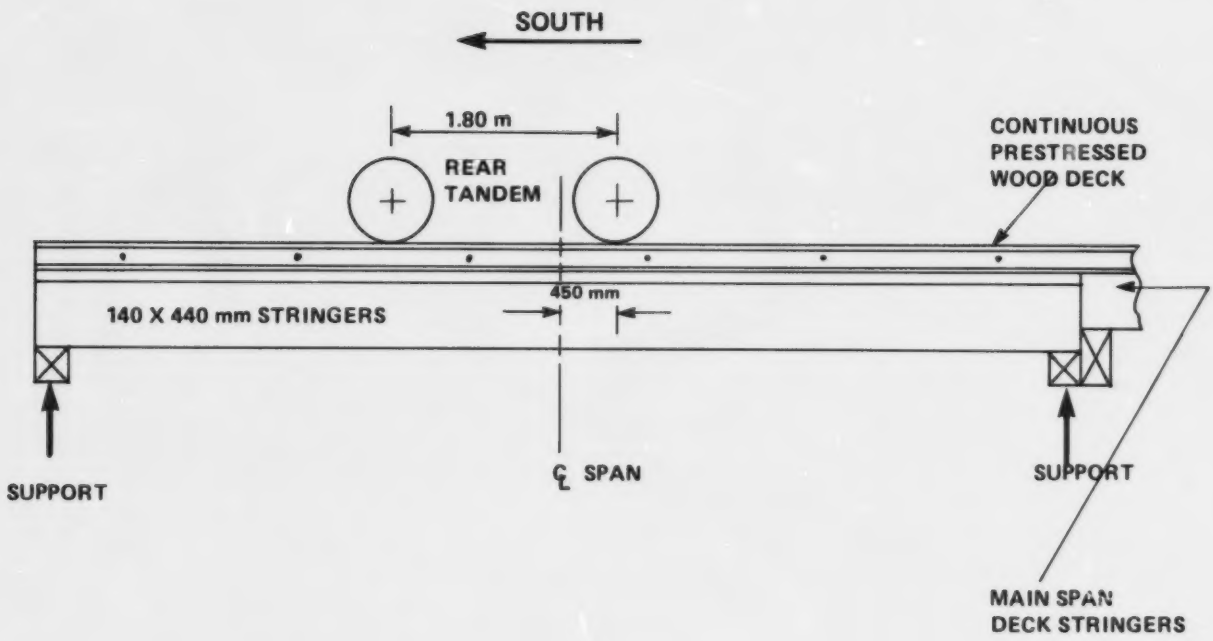


Figure 50/ Longitudinal Vehicle Position, South Approach Span

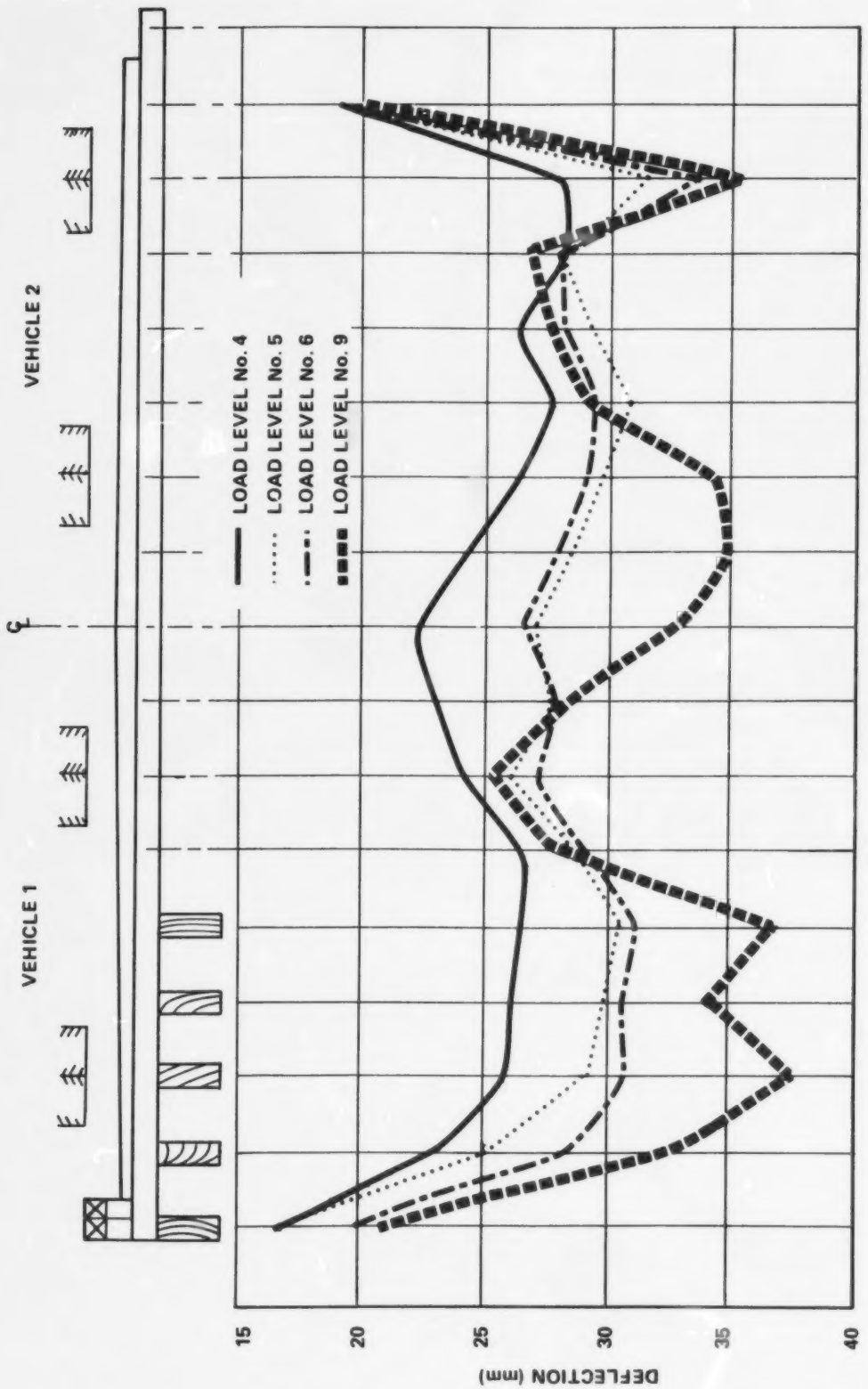


Figure 51/ Midspan Deflections, Test Results, South Approach Span

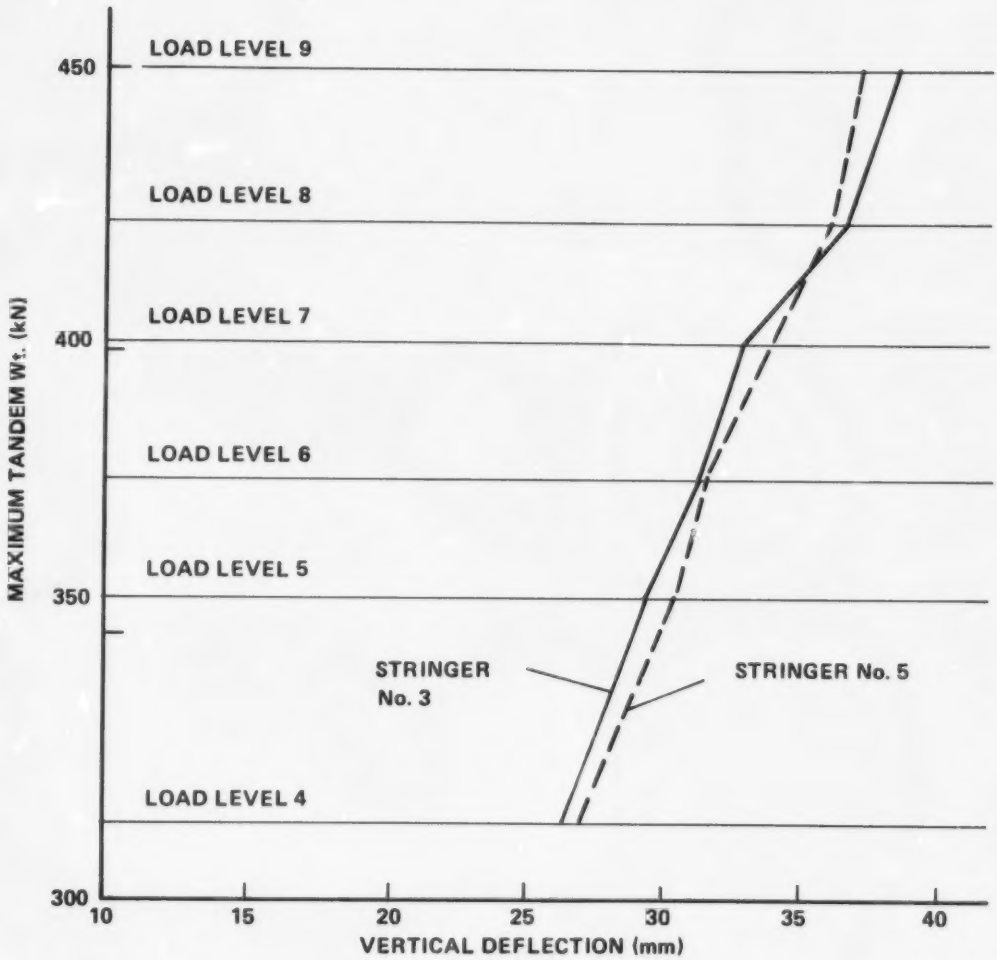


Figure 52/ Load Versus Deflection, South Approach Span

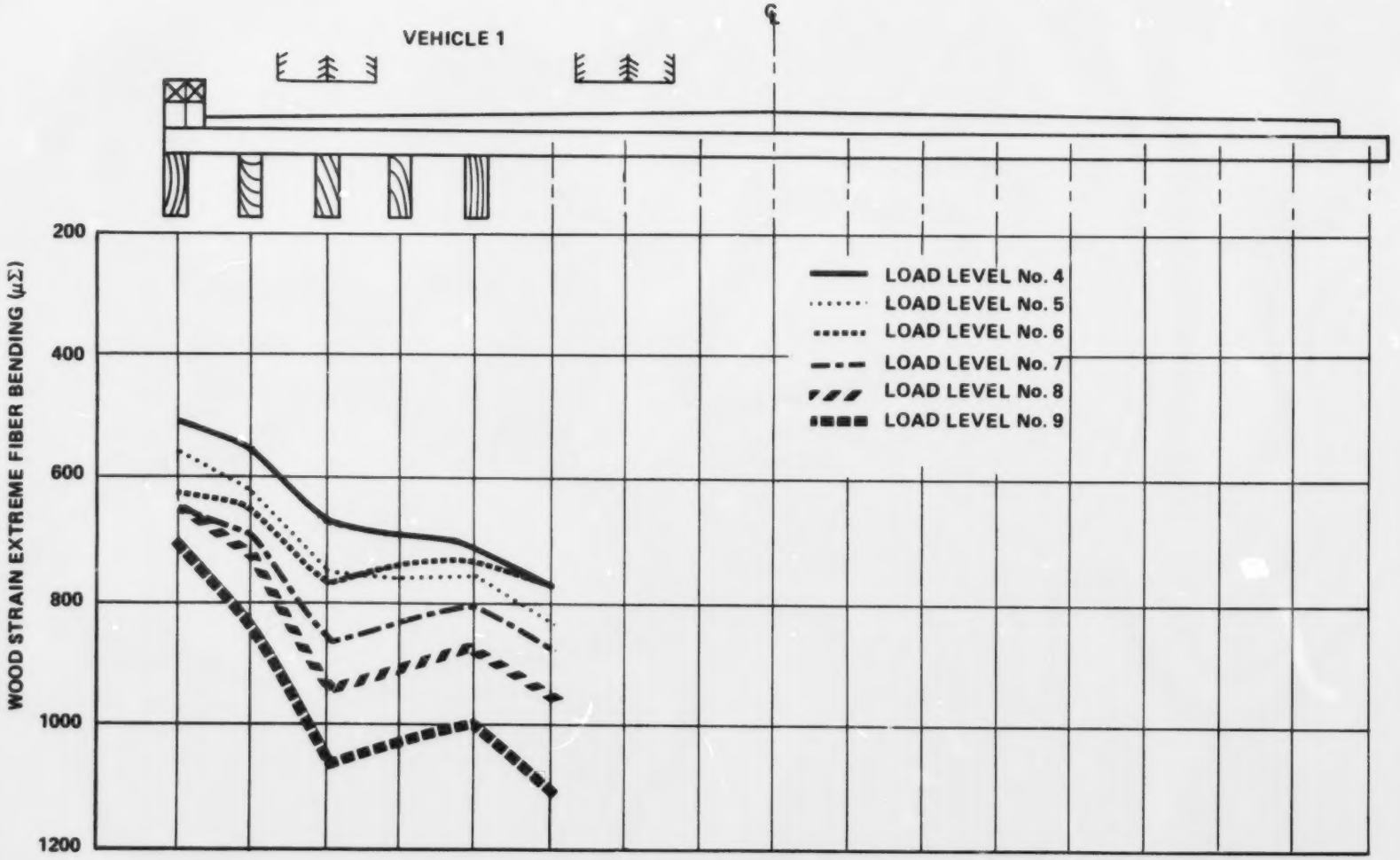


Figure 53/ Midspan Strains, Test Results, South Approach Span

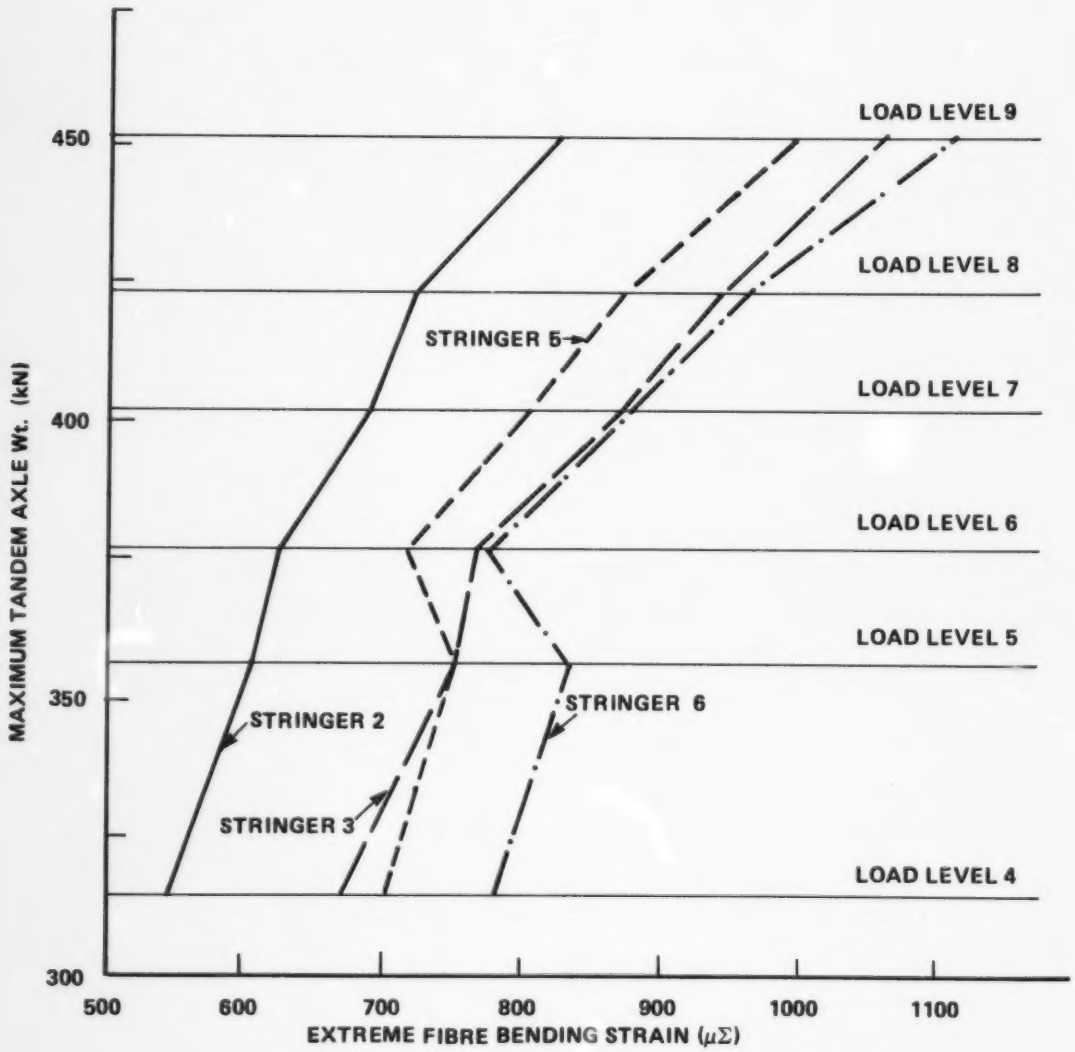
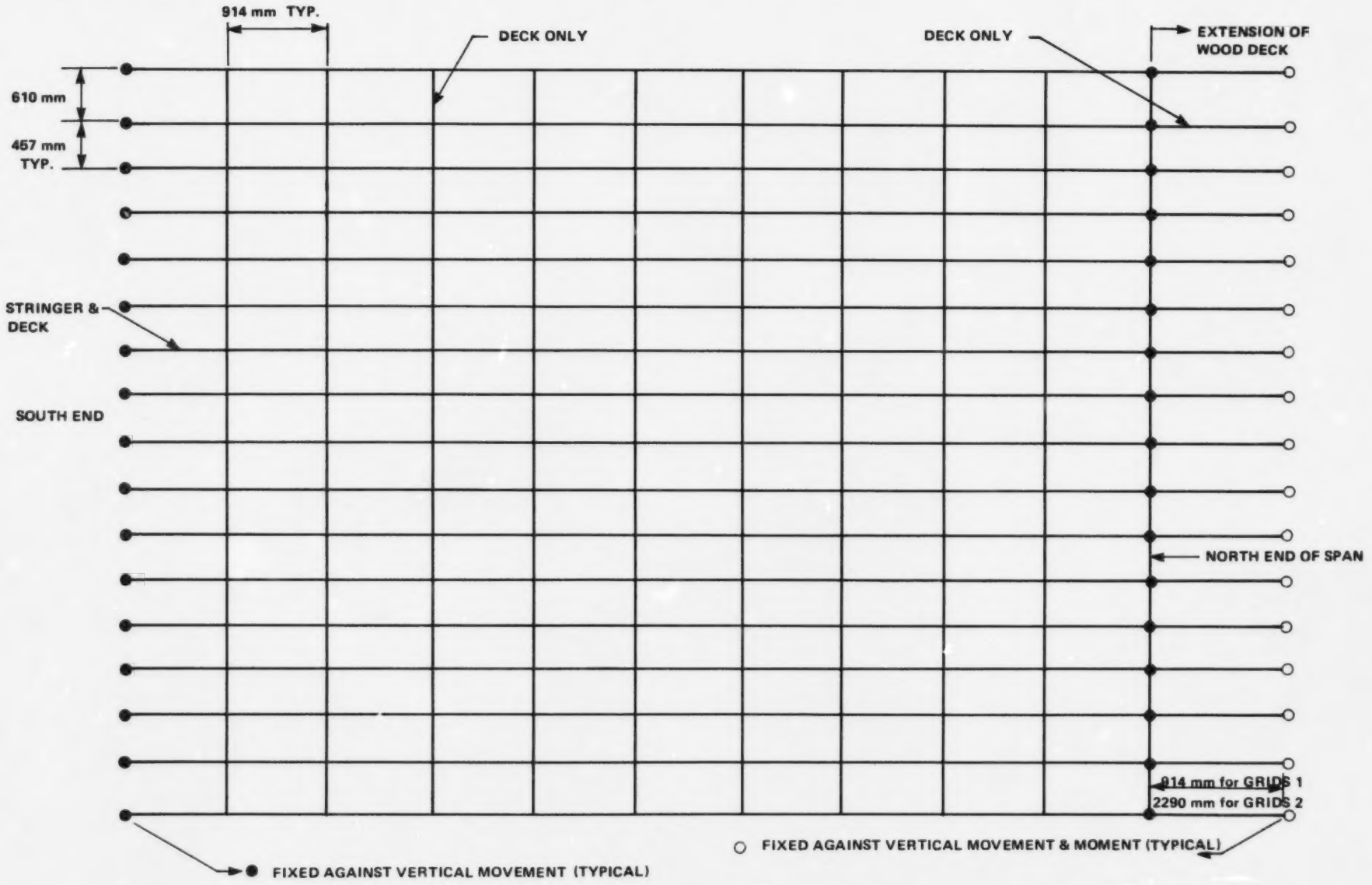


Figure 54/ Load Versus Strains, South Appraoch Span



F igure 55/ Grid Analogy – South Approach Span

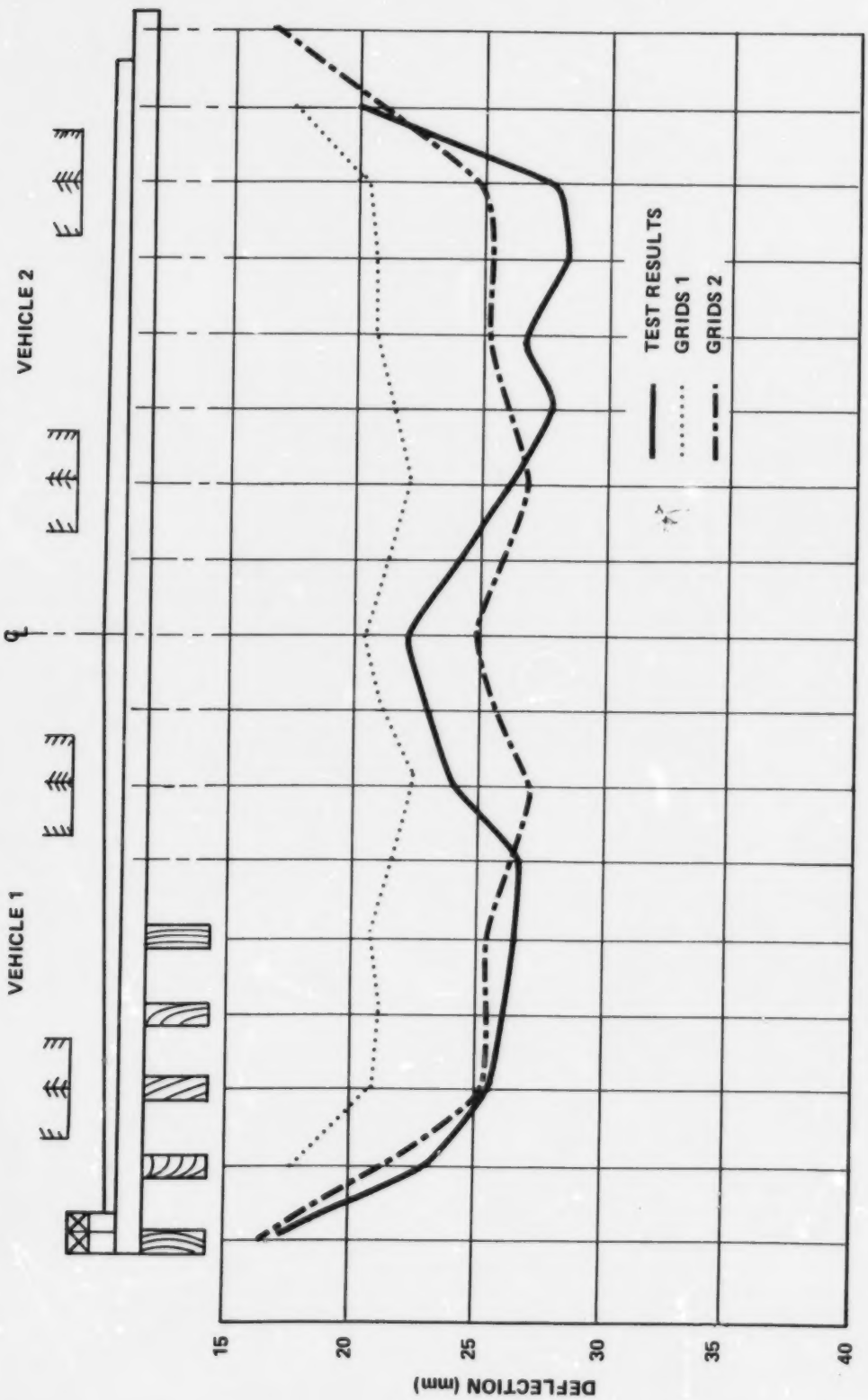


Figure 56/ Analytical vs. Experimental, South Approach Span Load Level 4

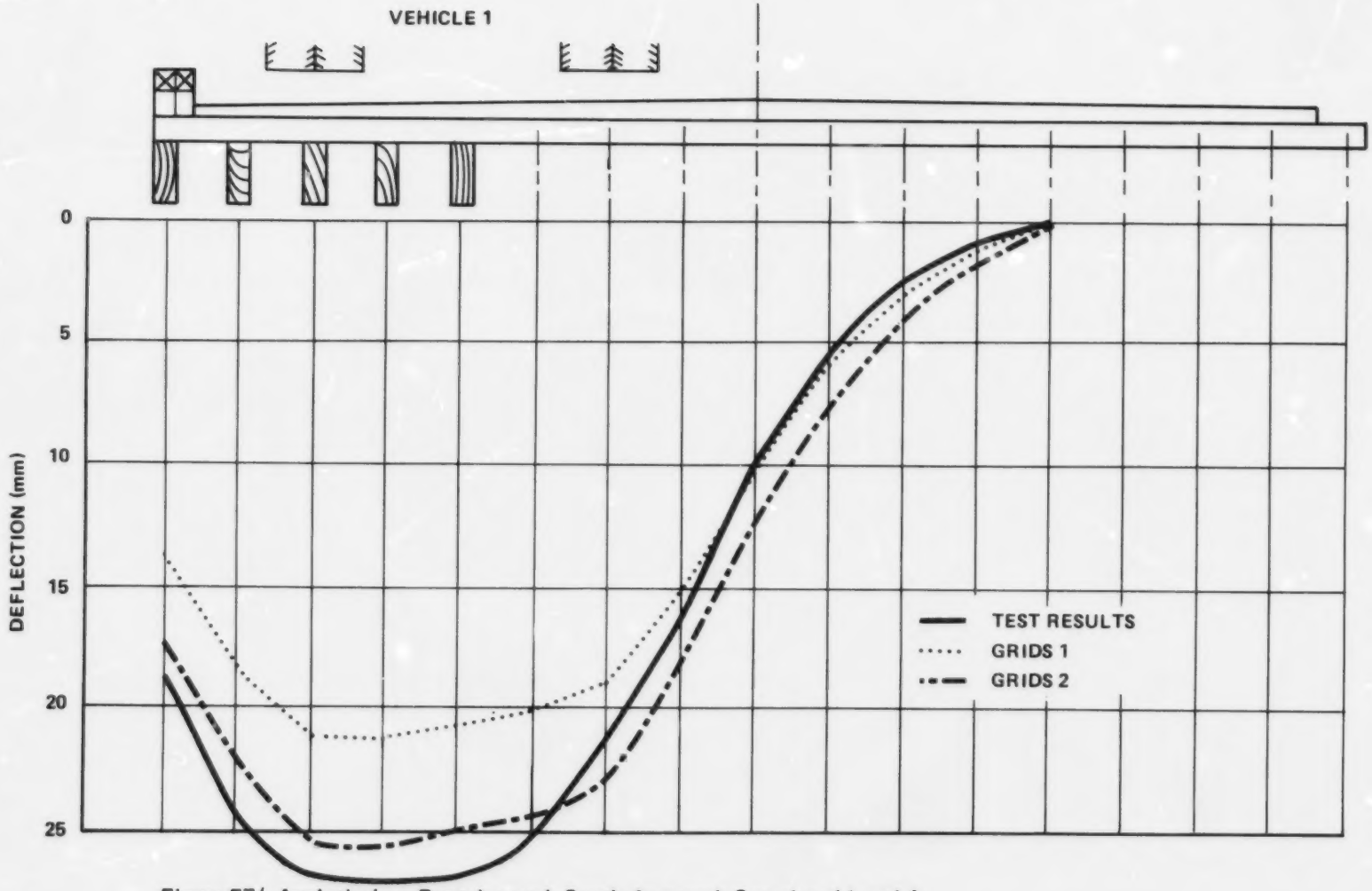


Figure 57/ Analytical vs. Experimental, South Approach Span Load Level 4

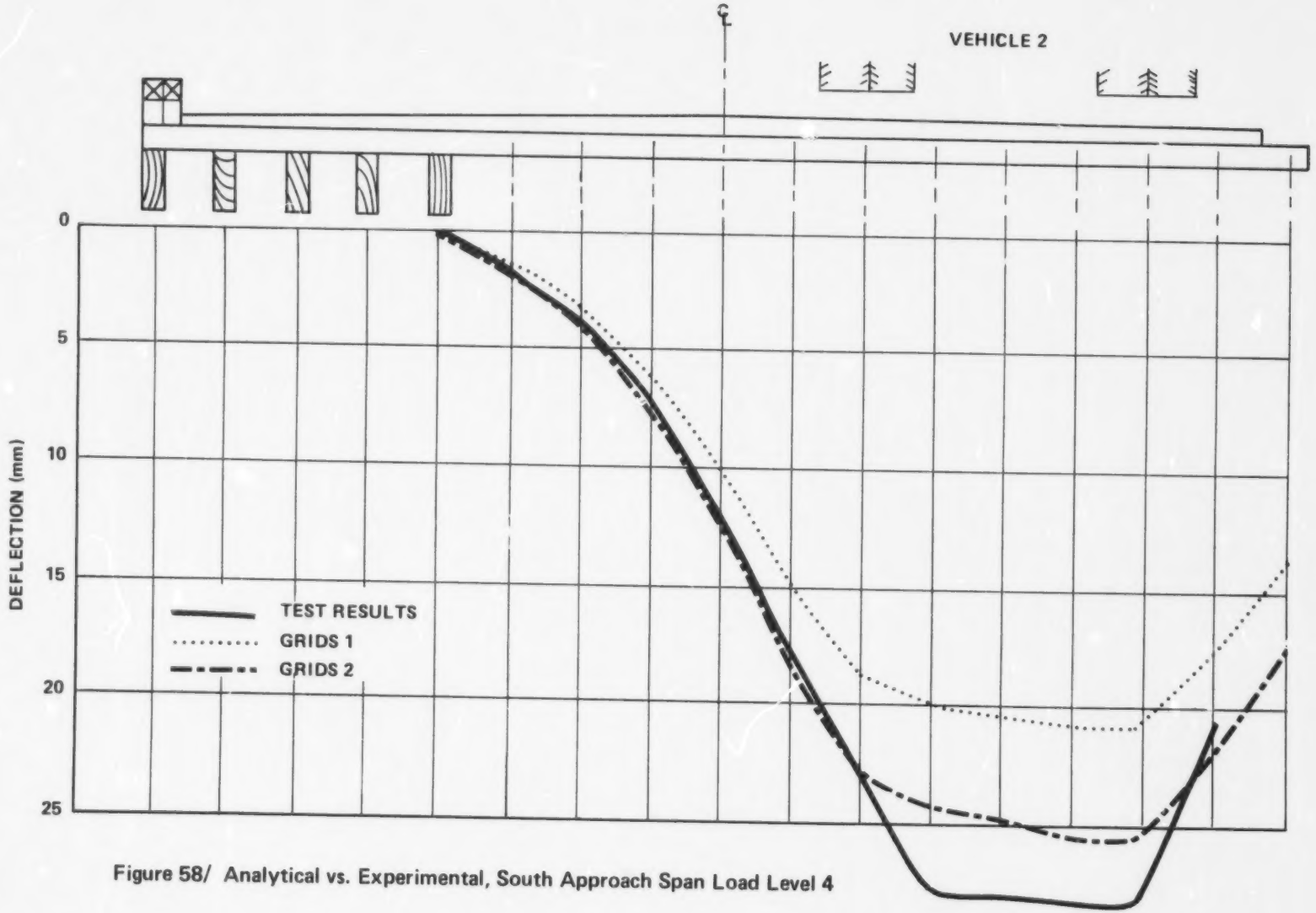


Figure 58/ Analytical vs. Experimental, South Approach Span Load Level 4

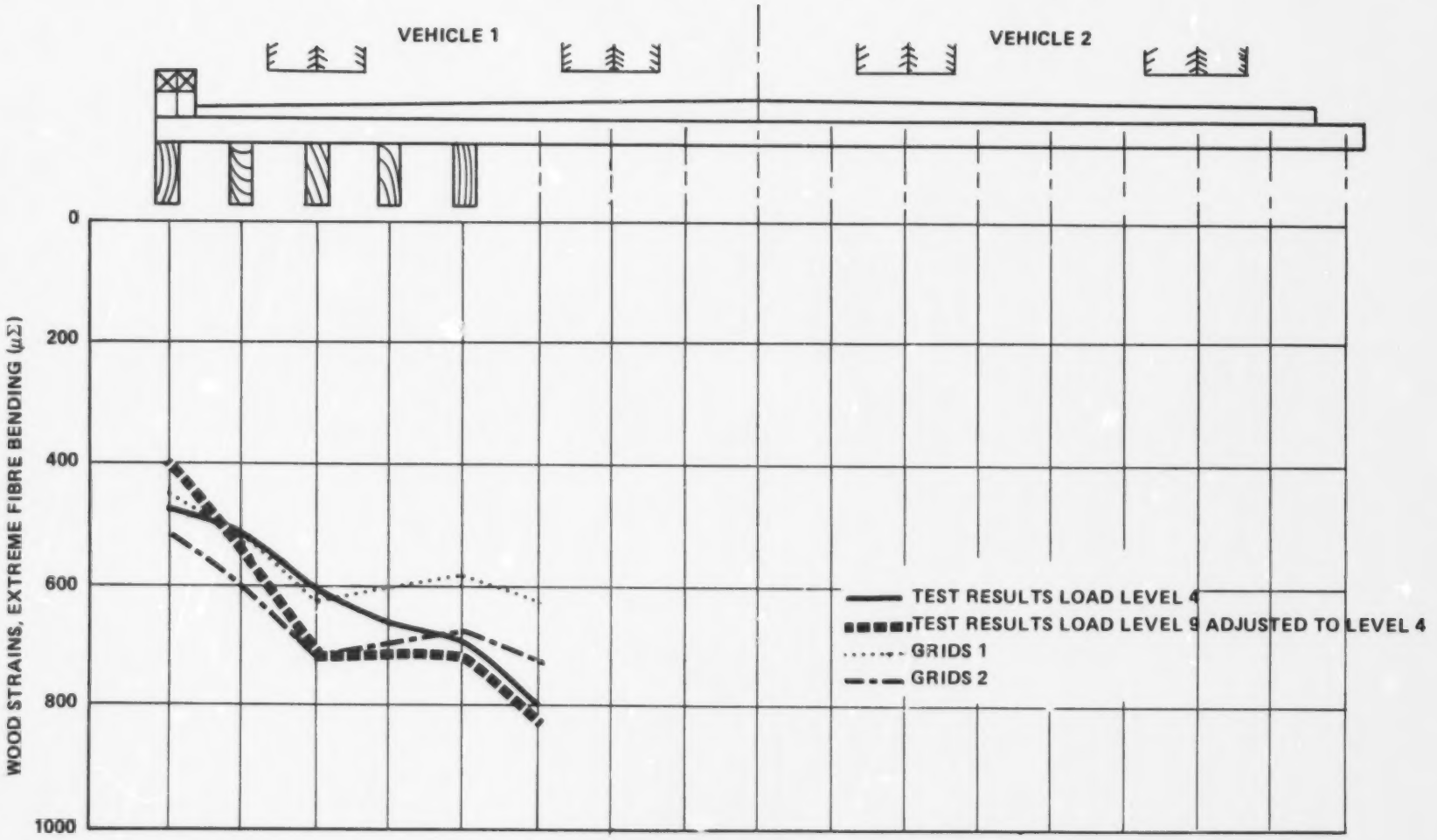
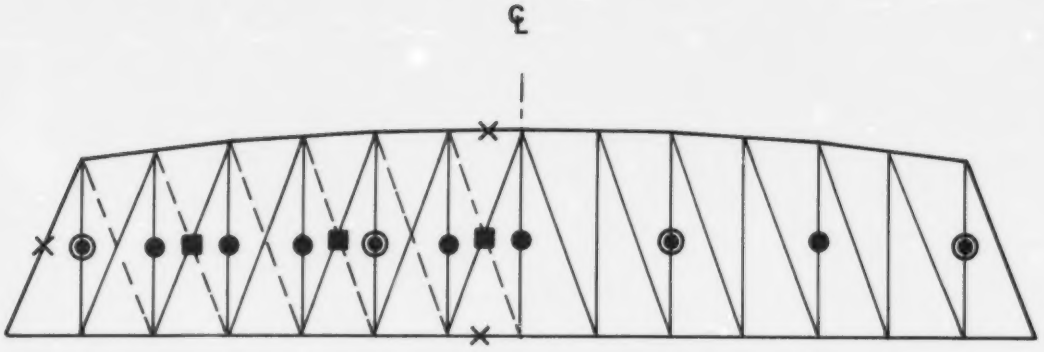


Figure 59/ Analytical Vs. Experimental, South Approach Span



● ELECTRICAL RESISTANCE STRAIN GAUGES – ONE GAUGE PER BAR

○ SIMILAR INSTRUMENTATION ON EAST TRUSS

× DEMOUNTABLE STRAIN GAUGES – TWO GAUGES PER LAMINATE (SEE FIGURE 62a)

■ DEMOUNTABLE STRAIN GAUGES – TWO GAUGES PER LAMINATE (SEE FIGURE 62 b)

Figure 60/ Instrumentation Layout – Main Span Truss – West Side

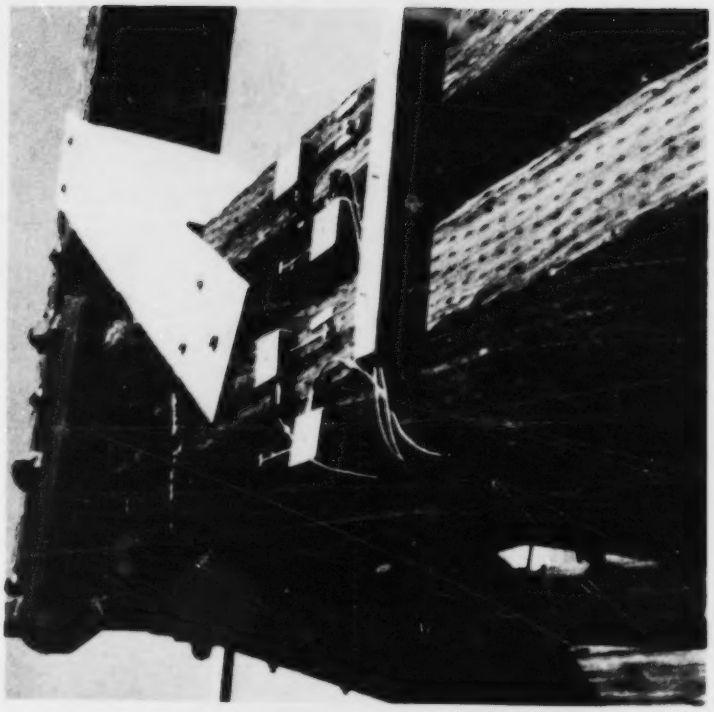
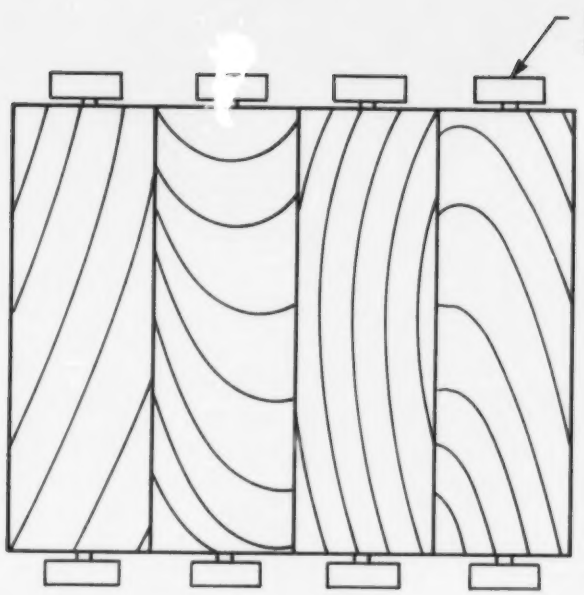
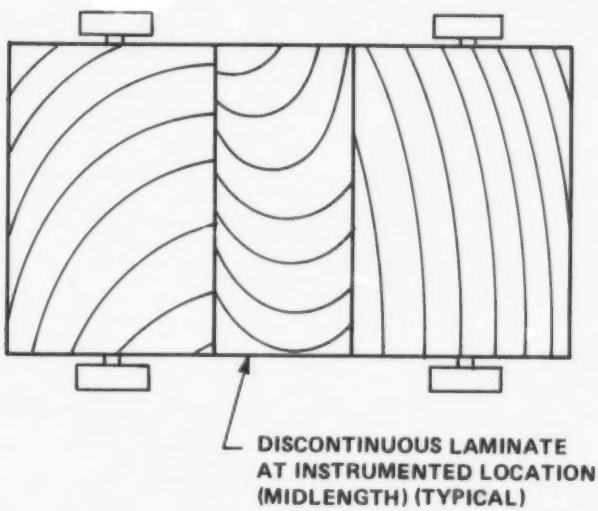


Figure 61/ Demountable Strain Gauges on Wood Members



(a) TYPICAL GAUGE LOCATIONS, FOUR-LAMINATED MAIN CHORDS



(b) TYPICAL GAUGE LOCATIONS, THREE-LAMINATE PRIMARY DIAGONALS

Figure 62/ Typical Demountable Strain Gauge Locations

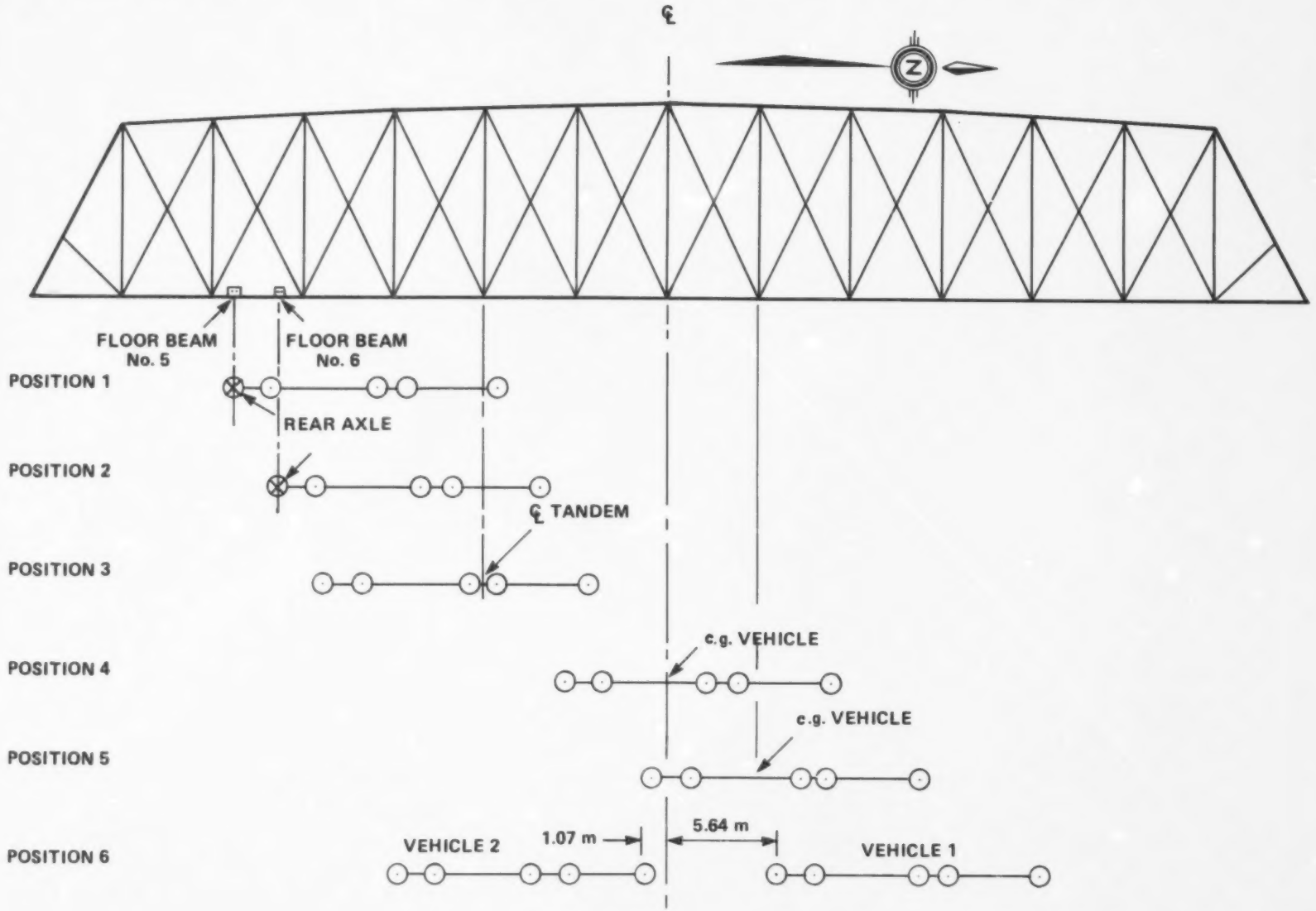


Figure 63/ Longitudinal Vehicle Positions – Main Span Truss

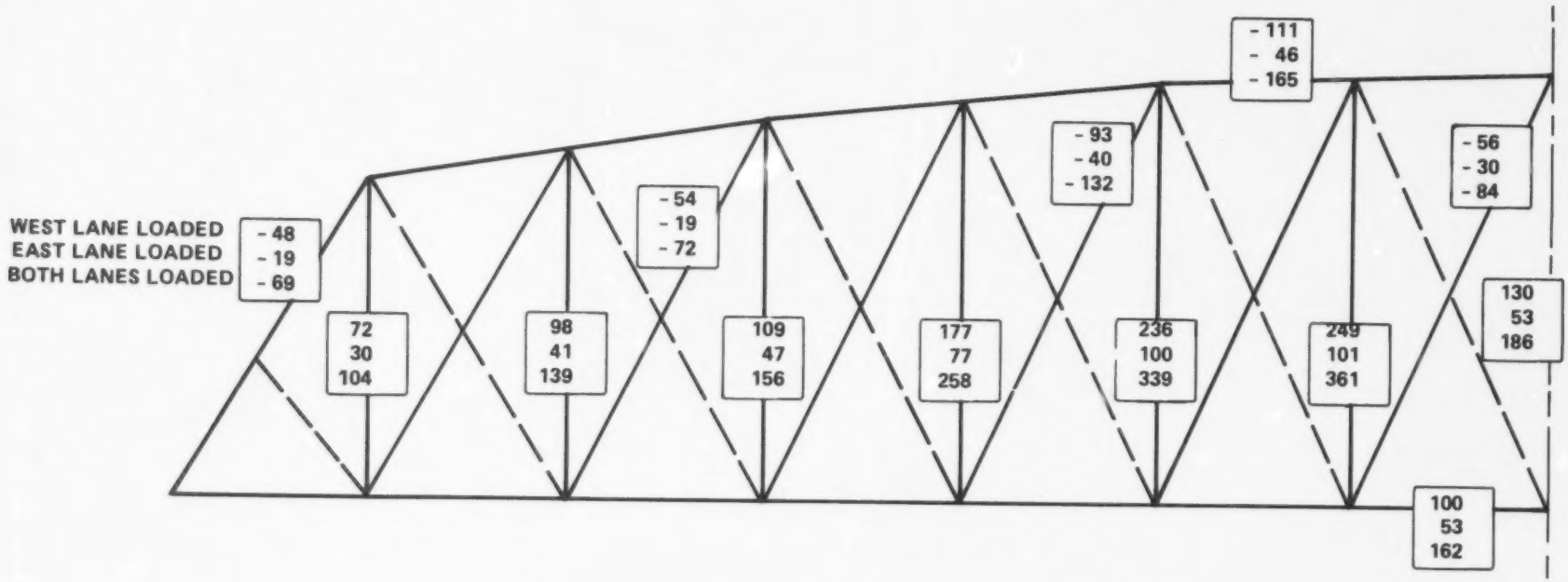


Figure 64/ Experimental Strains ($\mu\Sigma$), West Truss, Load Level 4, Longitudinal Position 3

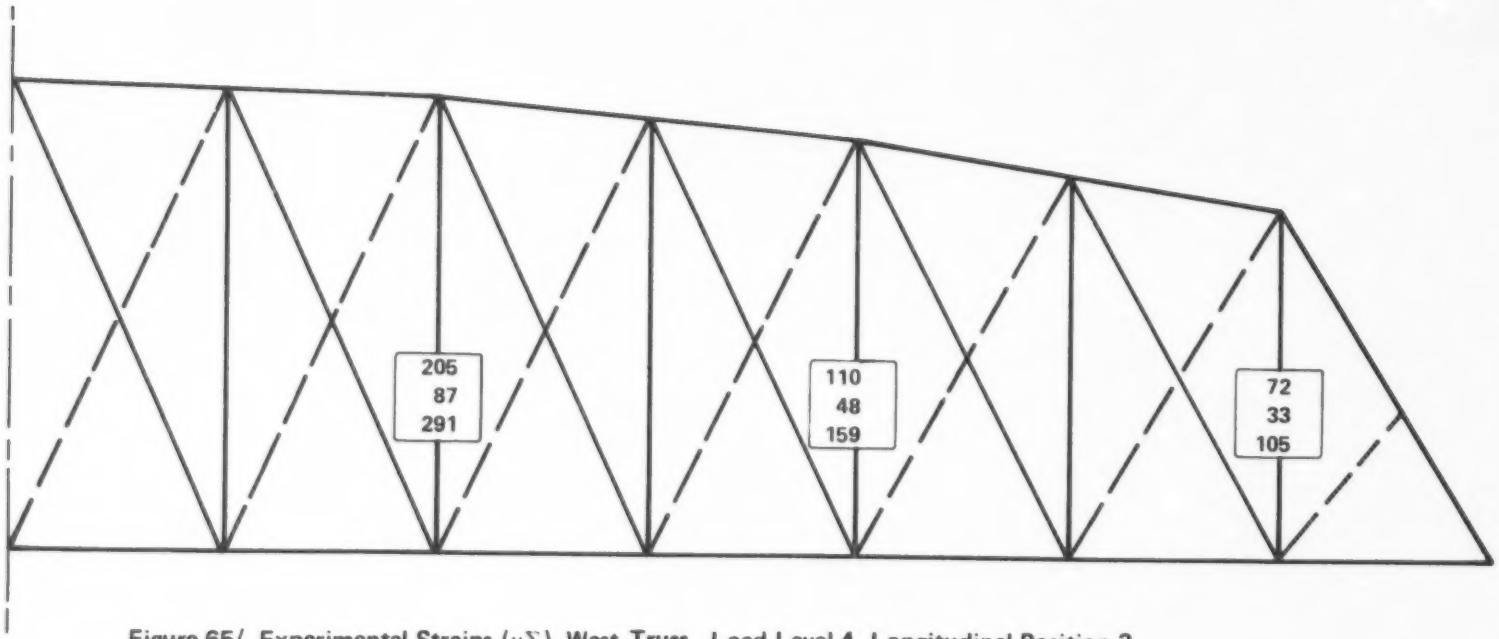


Figure 65/ Experimental Strains ($\mu\Sigma$), West Truss, Load Level 4, Longitudinal Position 3

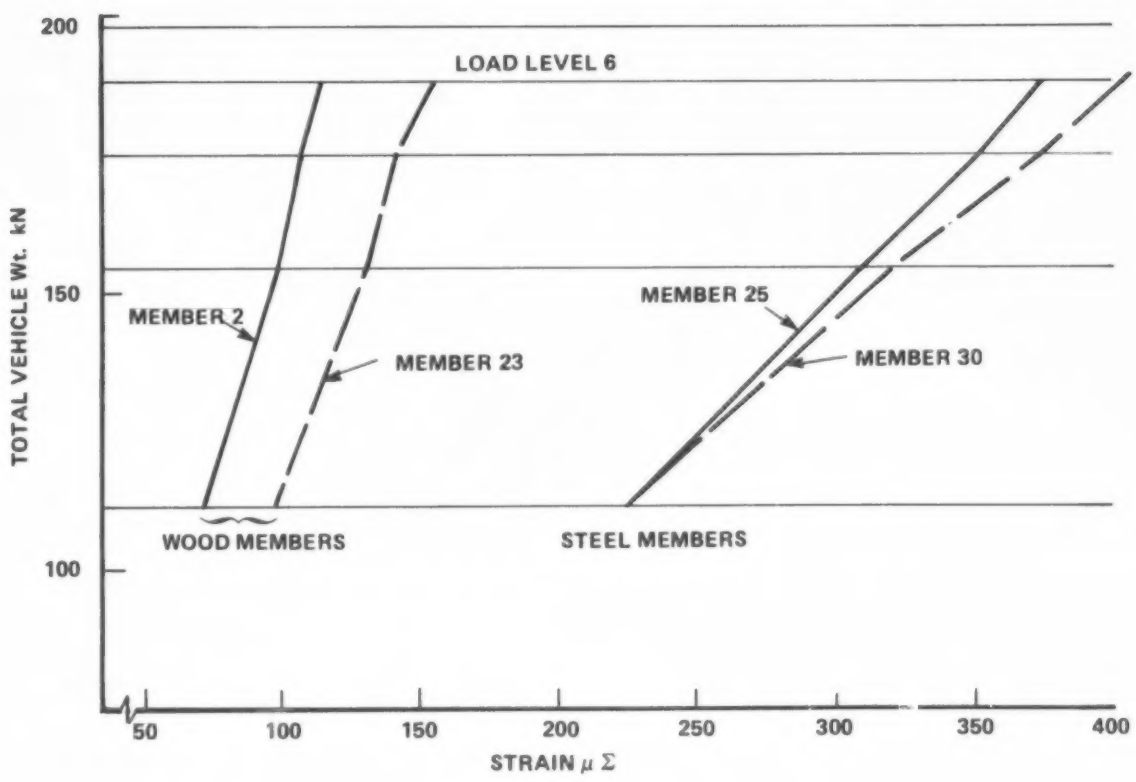


Figure 66/ Strain Versus Load, Main Span Truss

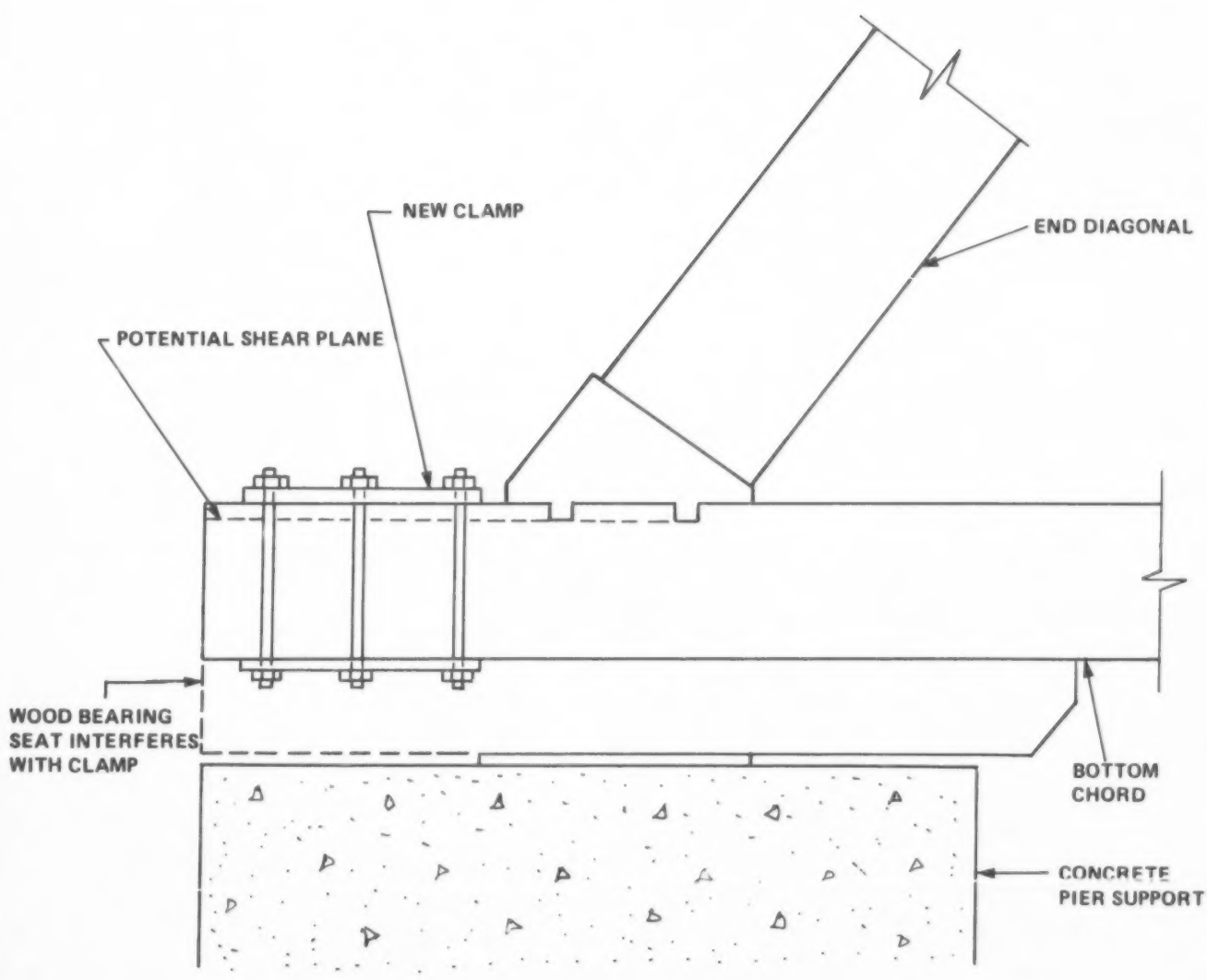


Figure 80/ Clamp Reinforcement of Shear Plane in Bottom Chord

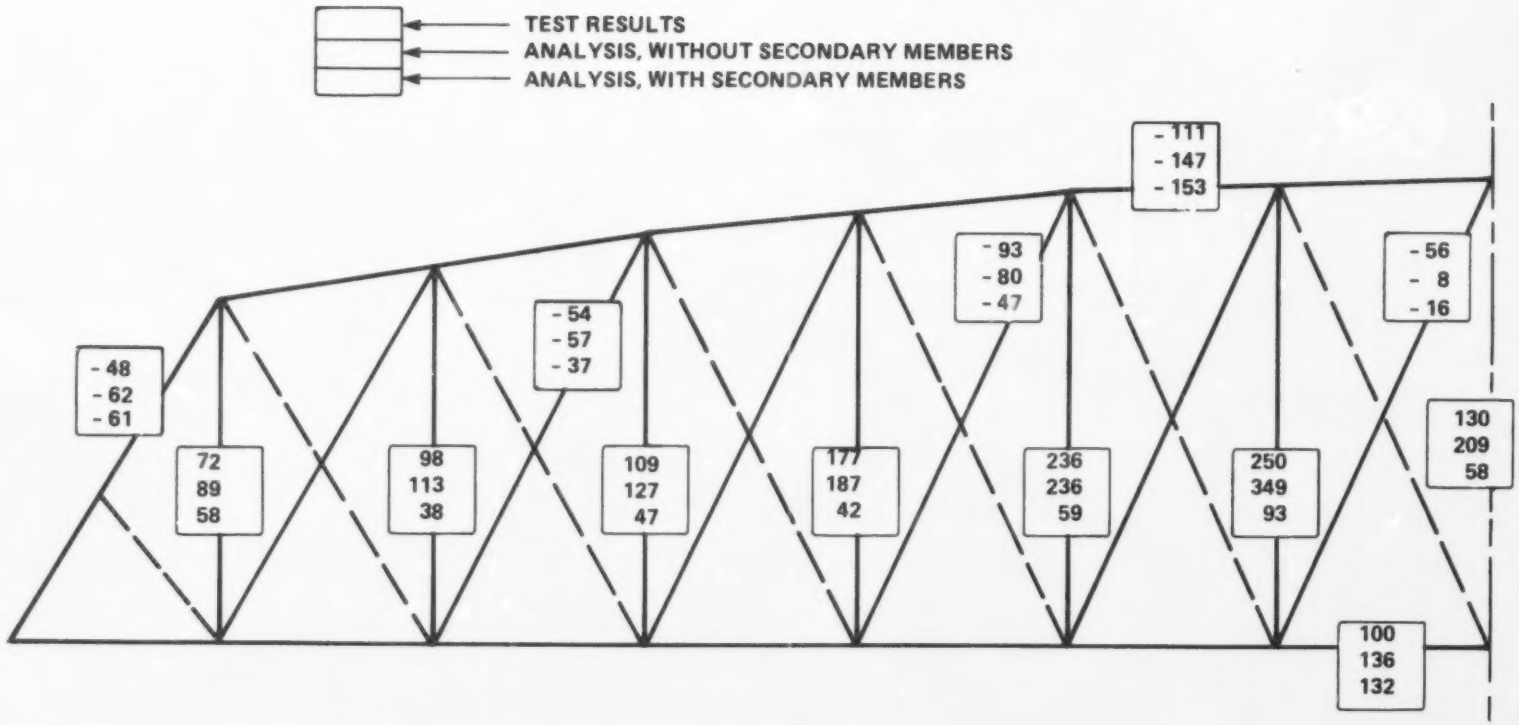


Figure 67/ Analytical vs. Experimental ($\mu\epsilon$), Load Level 4, Longitudinal Position 3, East Lane Loaded

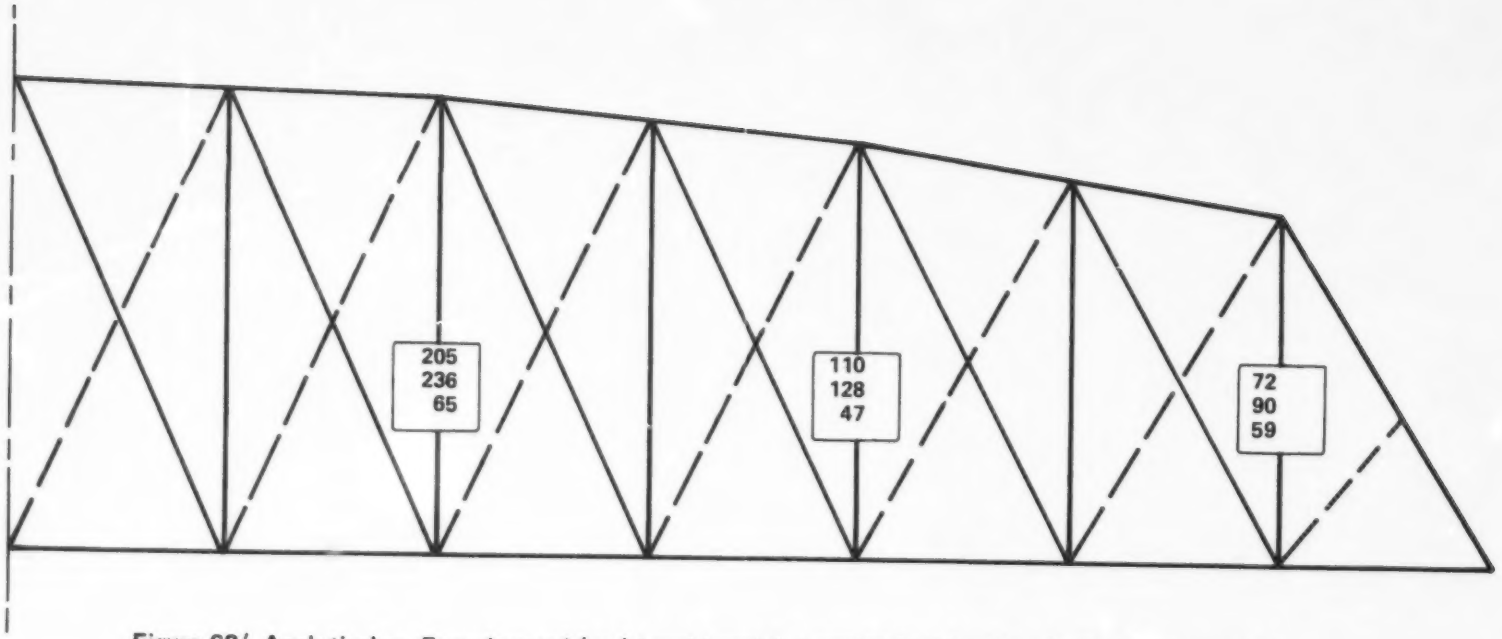


Figure 68/ Analytical vs. Experimental ($\mu\epsilon$) Load Level 4, Longitudinal Losition 3, East Lane Loaded

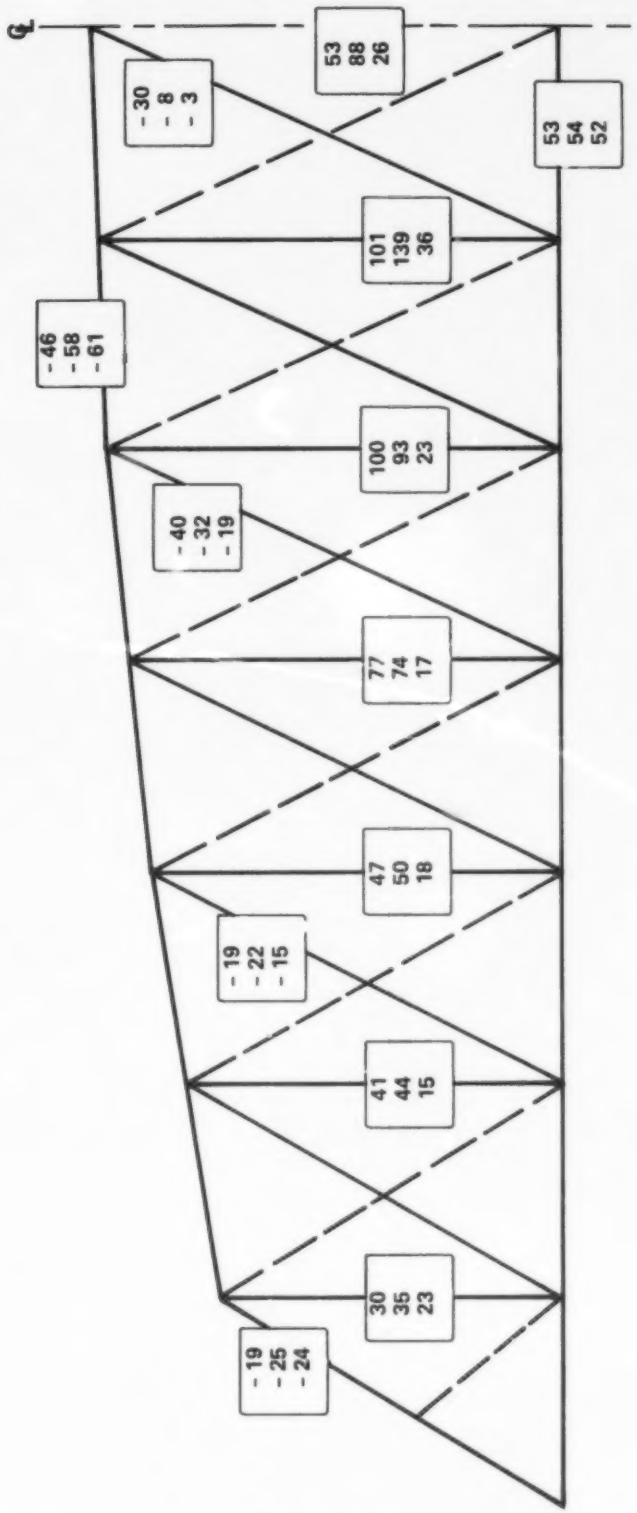


Figure 69/ Analytical vs. Experimental ($\mu\epsilon$) Load Level 4, Longitudinal Position 3, West Lane Loaded

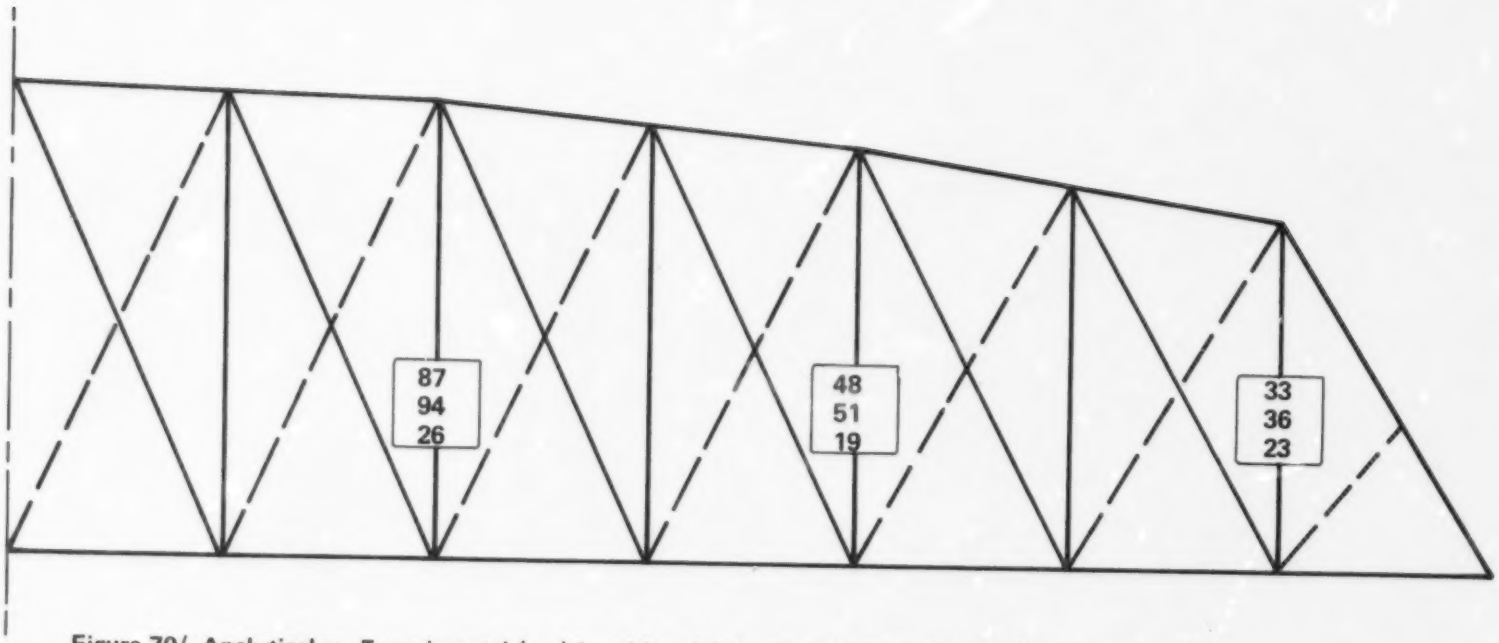


Figure 70/ Analytical vs. Experimental ($\mu\epsilon$) Load Level 4, Longitudinal Position 3, West Lane Loaded

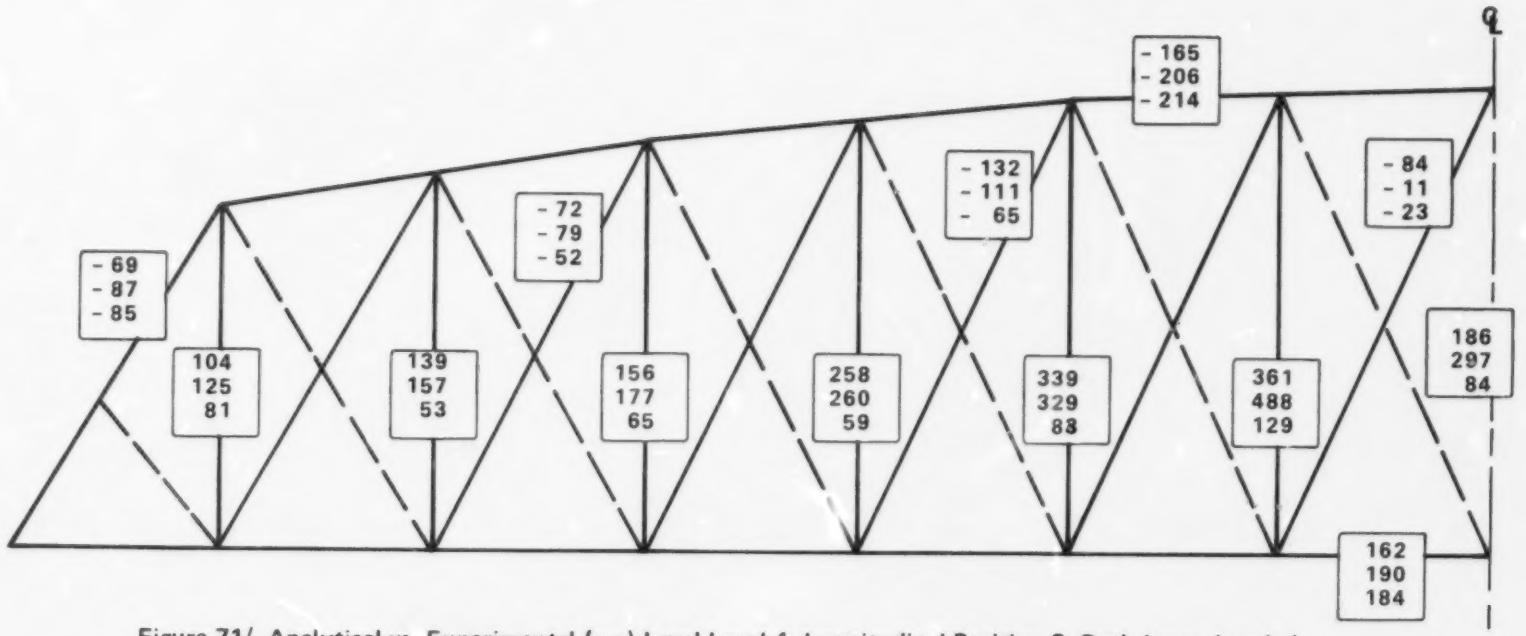


Figure 71/ Analytical vs. Experimental ($\mu\epsilon$) Load Level 4, Longitudinal Position 3, Both Lanes Loaded

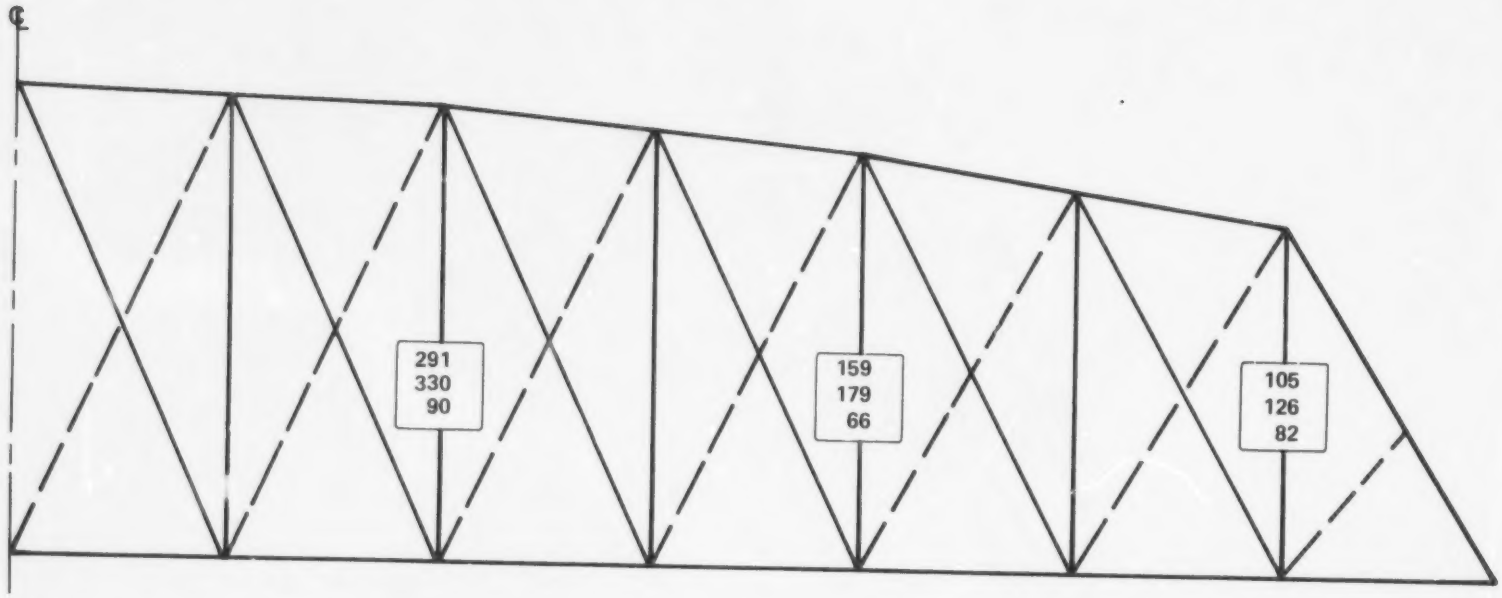


Figure 72/ Analytical vs. Experimental ($\mu\epsilon$), Load Level 4, Longitudinal Position 3, Both Lanes Loaded

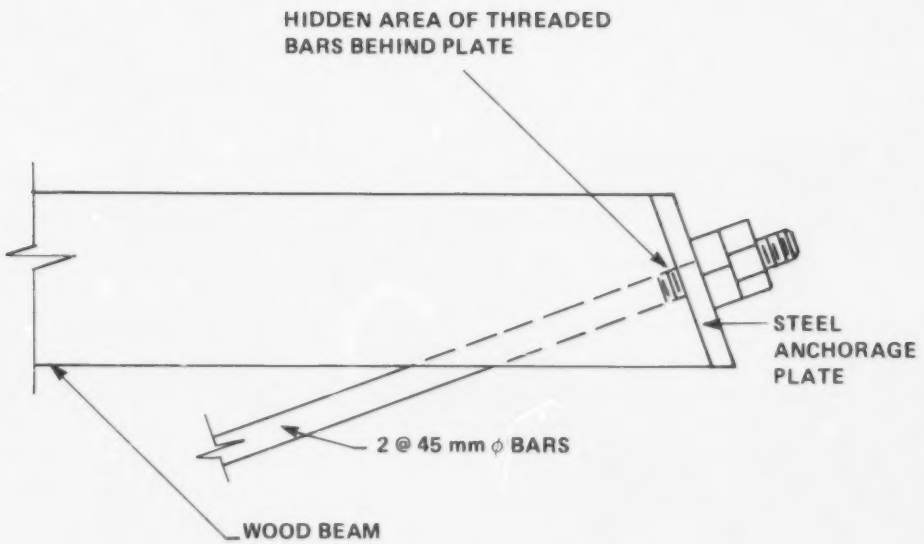


Figure 73/ Kingpost Floor Beam, Steel Bar Anchorage

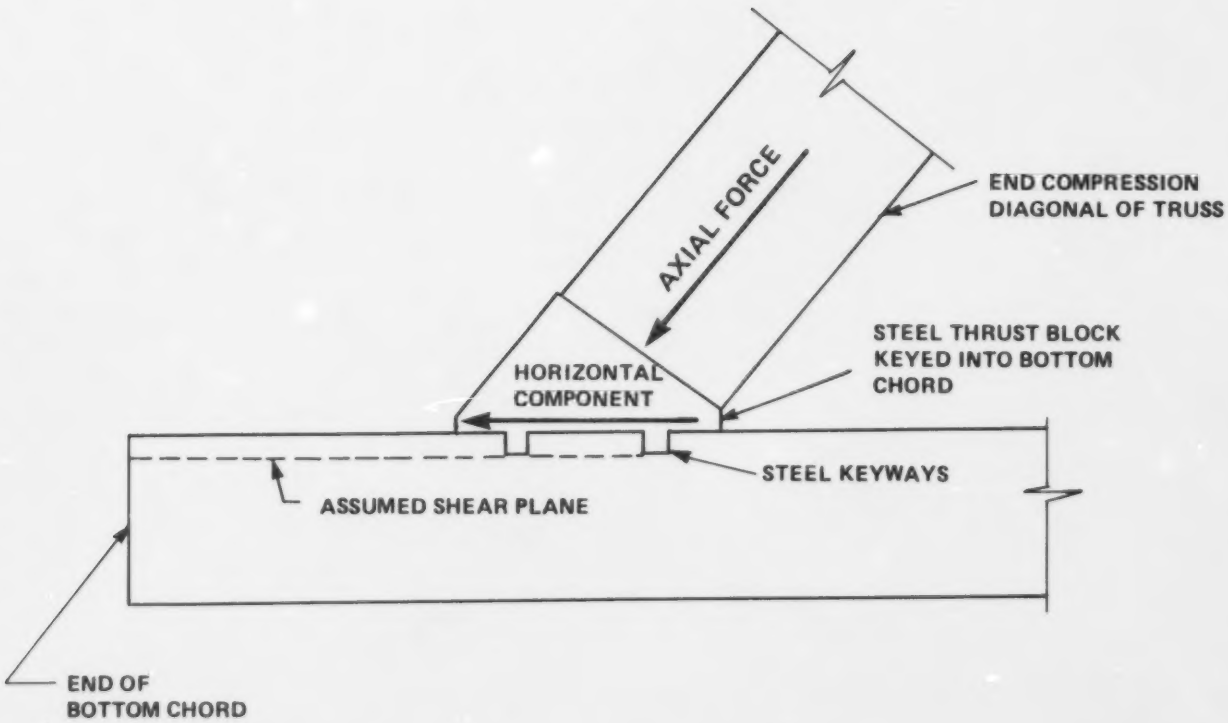


Figure 74/ Horizontal Shear at End of Bottom Chord

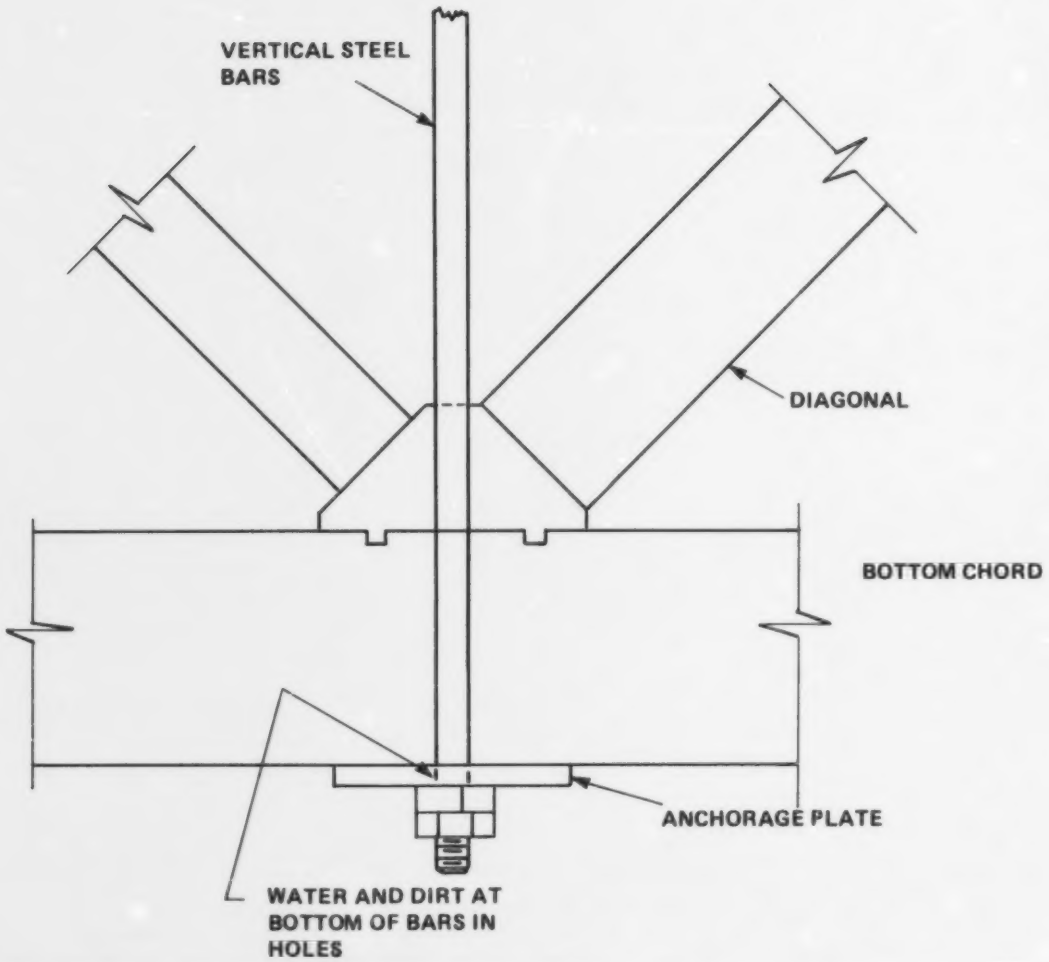


Figure 75/ Main Truss, Steel Bar Anchorage

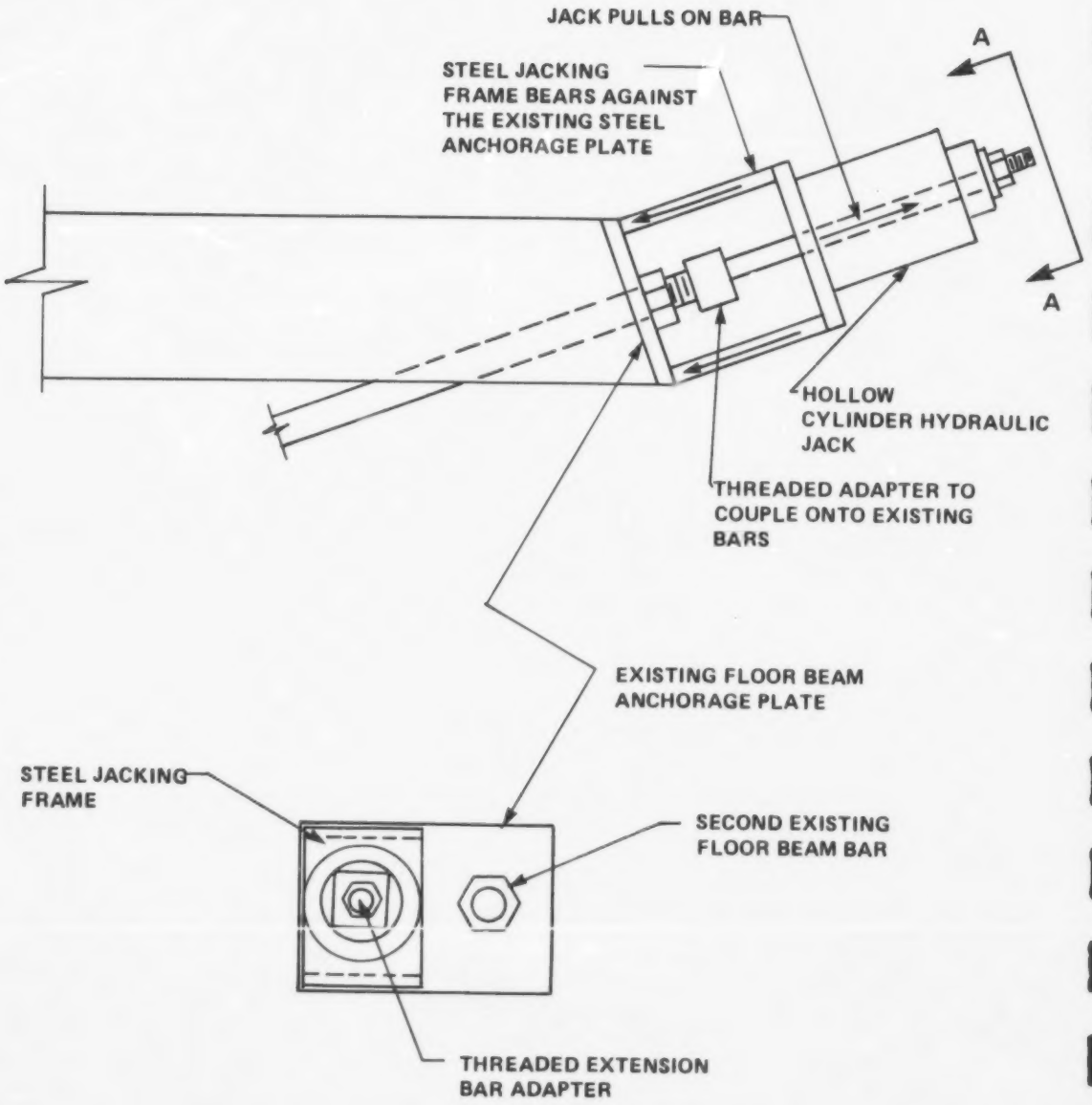

JACK PULLS ON BAR
STEEL JACKING FRAME BEARS AGAINST THE EXISTING STEEL ANCHORAGE PLATE
A
A
HOLLOW CYLINDER HYDRAULIC JACK
THREADED ADAPTER TO COUPLE ONTO EXISTING BARS
EXISTING FLOOR BEAM ANCHORAGE PLATE
STEEL JACKING FRAME
SECOND EXISTING FLOOR BEAM BAR
THREADED EXTENSION BAR ADAPTER
SECTION: A - A

Figure 76/ Suggested Jacking System for Floor Beams

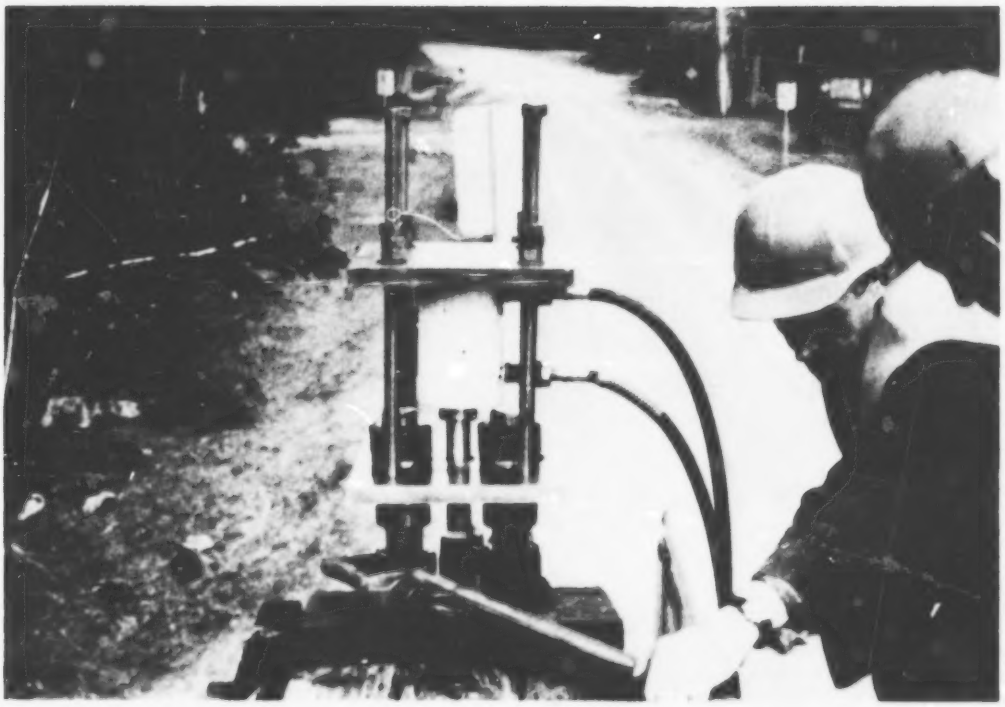


Figure 77/ Hydraulic Jacking System, Sydney Creek

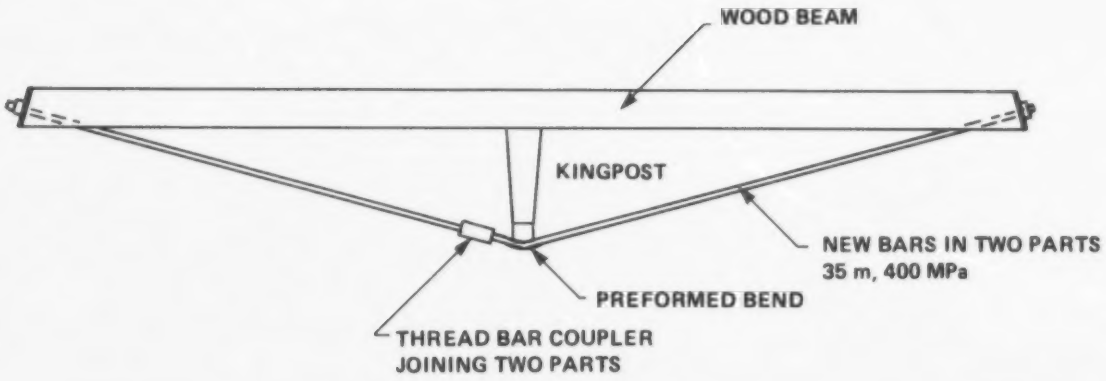


Figure 78/ Two-Part Replacement of Kingpost Floor Beam Bars

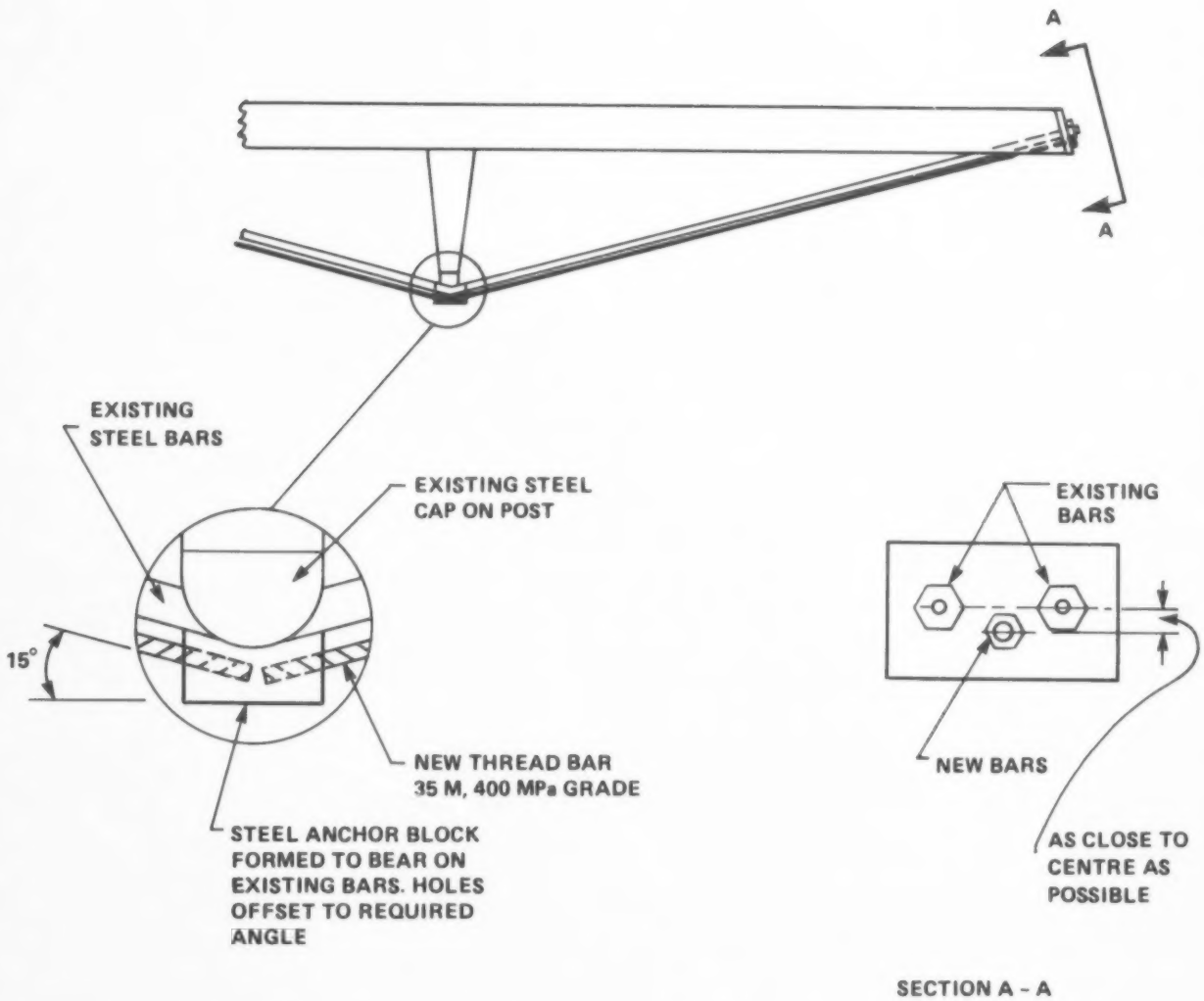


Figure 79/ Addition of New Superimposed Floor Beam Bar

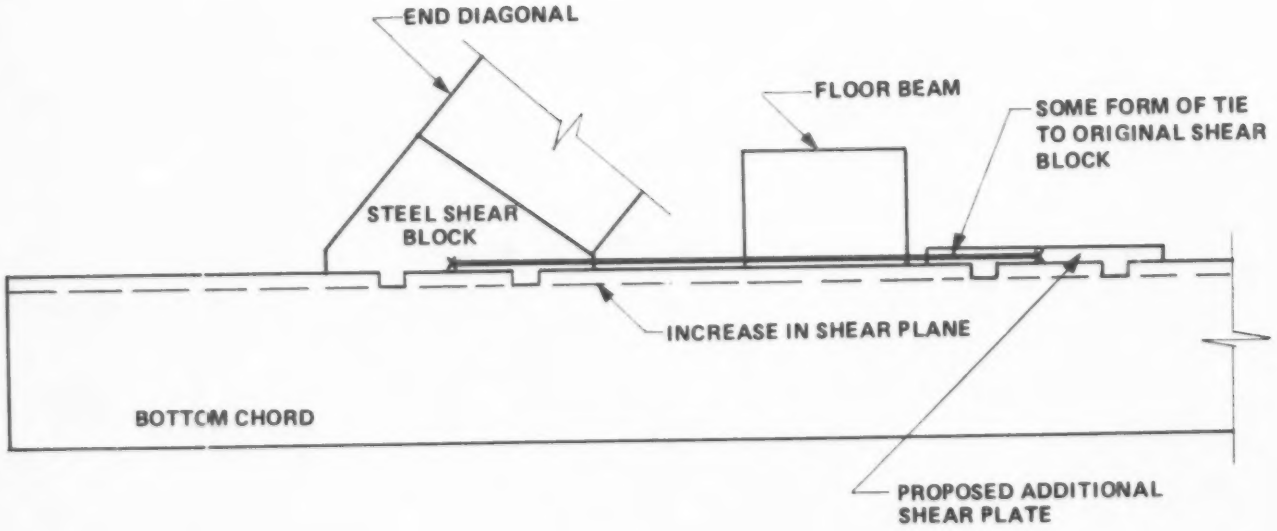


Figure 81/ Additional Shear Plate Strengthening

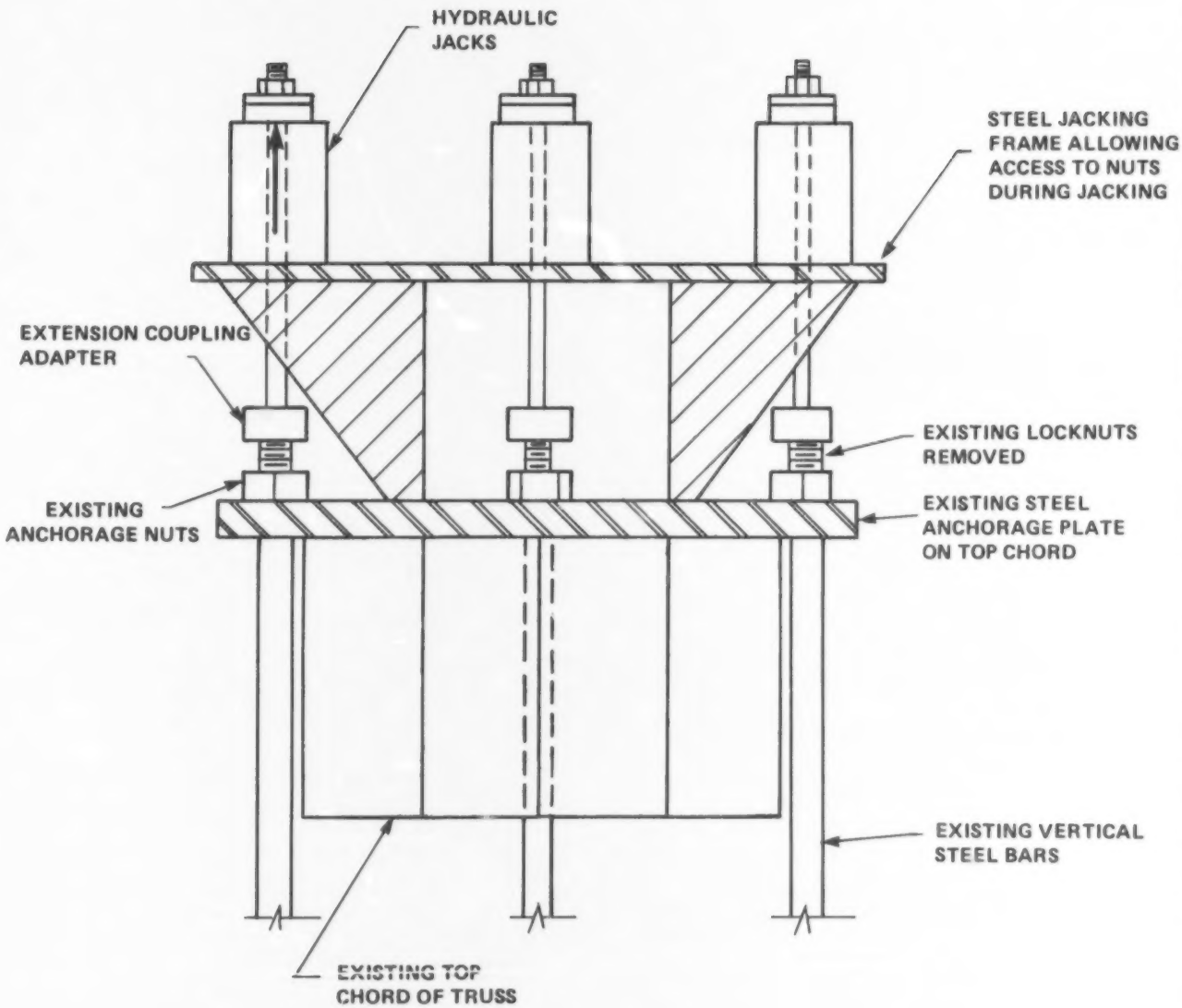

HYDRAULIC JACKS
STEEL JACKING FRAME ALLOWING ACCESS TO NUTS DURING JACKING
EXTENSION COUPLING ADAPTER
EXISTING LOCKNUTS REMOVED
EXISTING ANCHORAGE NUTS
EXISTING STEEL ANCHORAGE PLATE ON TOP CHORD
EXISTING VERTICAL STEEL BARS
EXISTING TOP CHORD OF TRUSS

Figure 82/ Schematic of Proposed Jacking System, Main Truss Bars